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THE MANHATTAN SUSPENSION BRIDGE over the East River at New York will be delayed possibly a year or more by the action of the Board of Aldermen of New York City, who on July 14 defeated a resolution to appropriate $6,533,000 for the superstructure of the bridge. This bridge has been in hand for some years, and the foundations of the towers are practically completed. Full plans had been prepared for a wire-cable suspension bridge, but when the present Commisioner of Bridges of New York City, Mr. Gustav Lindenthal, M. Am. Soc. C. E., came into office he rejected the old plans and worked out a new design, which employs eye-bar chains in place of wire cables as the supporting element of the bridge. An extended discussion has been waged over the new plans, part of which discussion has appeared in the pages of Engineering News. The architectural design of the bridge was shown by several views (from the plans) in our issue of Feb. 19, 1903. At a public hearing before the Board of Estimate of New York City a number of well-known engineers protested against the appropriation of money for the construction of the bridge on Commisioner Lindenthal's plans, on the ground that the cost of such a bridge would exceed by several million dollars the cost of an equivalent bridge built with wire cables. This board passed the appropriation, however. The subsequent action of the Board of Aldermen now defeats the appropriation and unless the board reconsiders its action after its summer vacation, now begun, it will be impossible to call for bids or let contracts until favorable action is secured before another Board of Aldermen. We would call attention here to an editorial note on the discussion over the bridge plans, in our last issue (July 9). We advised that bids be asked on both types of construction, eye-bar-chain and wire-cable, in order that definite figures as to relative cost and time of completion may be available. This same course was recommended, subsequently, in the debate of the Board of Aldermen; but as it appeared that only a definite granting or refusal of the appropriation was possible, the board voted to refuse it.

THE JEROME PARK RESERVOIR continues to be the subject of much unhappy controversy. Some time ago Mayor Low requested Prof. Wm. H. Burr, John R. Freeman and Rudolph Hering to report on the engineering questions at issue, which involve changes in the design of the embankments and the water tightness of both the banks and bottom of the reservoir. July 15 Mayor Low sent a letter to Professor Burr, as chairman of the special commission, asking the commission to hasten its report. As the letter involves several points of wide interest, and is brief, we reprint it in full, as follows:

Dear Sir: The resignation of Mr. William R. Hill, the Chief Engineer of the Aqueduct Commission, on account of illness, places upon the commissioners the duty of selecting a new chief engineer to carry to completion the important work committed to their charge. The commissioners, I am confident, will seek to appoint to this position a man who, by reason of character and ability, will deserve the confidence of the entire community. But even this, while essential, is not enough. It is of the utmost importance that the new chief engineer shall take charge of the work under such conditions as to be in no way embarrassed by the engineering controversies that have raged since the retirement of Mr. Fteley.

To this end I desire to point out to you that an early report from your Commission on the engineering questions referred to it is very necessary. Any new chief engineer worthy of the trust may be relied upon to work out the details. Indeed, he must do so, for the responsibility will be his and no one else's; but any suitable appointee will be greatly aided if your Commission can indicate the general method of treatment proper, in your judgment, to be pursued as to the various matters in dispute that have been referred to you.

The proper treatment of the bottom of the reservoir is perhaps the most pressing question of all, and, therefore, the one upon which the most detailed suggestion is necessary at your hands, in order to avoid delay. I wish, therefore, to urge upon you as earnestly as I may that the report of your Commission on all the matters referred to it be placed in the hands of the Aqueduct Commissioners, if possible, on or before Saturday, July 18.

With your report in hand, the Aqueduct Commission can proceed without delay to select its chief engineer, and arrangements can be promptly made to carry the work forward with renewed energy.

THE PROPOSED PASSAIC VALLEY DISTRICT sewer in its relation to the pollution of New York Bay has been reported on by a number of engineers. Their reports, as recently stated in these columns, were so favorable to the project that the attorney-general and the governor of New Jersey advised the Passaic Valley District Commission that it was at liberty to proceed under the recent statutory act authorizing the work. Among the engineers who reported, as just mentioned, was Gen. Henry M. Roberts. He bases his opinion that the proposed outlet will not cause a nuisance in the waters under the jurisdiction of New York State, upon the following figures:

The average ebb tide of the Hudson River at 39th St. is about 7,000,000,000 cu. ft., and the ebb tide at the Narrows, which includes the East River, is about 13,800,000,000 cu. ft., or twice as much as the Hudson alone.

The full capacity of the proposed sewerage system is about 330,000,000 gallons, or about 44,000,000 cu. ft. The sewage will not be evenly distributed over the twenty-four hours, and so, to be perfectly safe, let us suppose that the total amount of daily sewage has to be disposed of by one ebb tide and that tide equal to the ebb tide of the Hudson River at 39th St.

We would then have 44,000,000 cu. ft. of sewage to be carried away by 7,000,000,000 cu. ft. of water in the ebb tide, which would certainly be ample to prevent the sewage from proving a nuisance.

In this connection it may be well to notice the fact that New York Bay now receives more than double the amount of sewage that the proposed sewage system can possibly discharge into the bay, and that if the proposed plan were to-day in operation it would not add 15% to the sewage now discharged into the waters of New York harbor.

As to the danger from possible deposits in the harbor attention is called by Gen. Geo. S. Green, in another report, to the following:

Aside from the insignificance of the deposit of sewage, so far as New York interests are concerned, is the fact that the outfall or discharge of the sewage is on the westerly side of the main channel and there is not sufficient force to push or carry the sewage across the axis of the current in the channel, and, therefore, the sewage, with whatever mineral or other matter it may contain, will be carried entirely along the shore of New Jersey, except for the short stretch of the Narrows, one side of which is Staten Island, where the current is strongest and will carry such matter past Staten Island and out into the lower bay or the ocean.

A WATER FILTRATION PLANT FOR CLEVELAND, O., has been recommended by the State Board of Health, in a lengthy report recently laid before the local health board. The report was referred to a special committee, headed by Mr. J. Milton Dyer.

DYNAMITE EXPLODED July 14 at Morris Park Ave., New York, killing a watchman who was lighting some lanterns at the tool chest where the dynamite was kept.

A CONVENTION to oppose the proposed $101,000,000 bond issue for a barge canal in New York State has been called by the Chamber of Commerce of Rochester. The date set is July 21. John M. Ives is secretary of the organization just named.

THE MOST SERIOUS RAILWAY ACCIDENT of the week occurred on the Missouri Pacific Ry. near Lee's Summit, Mo., on July 12. An eastbound excursion train stalled on the main track was run into at high speed by a westbound fast mail train. Two persons were killed and several others injured.

AN EXPLOSION OF DYNAMITE occurred July 2 at the Whirlpool Rapids station of the Niagara Gorge Railway. The building was wrecked by the explosion, pieces of the building being hurled half a mile. A fire is said to have been started in the building by the dropping of a match in a toilet room, and the explosion followed when the fire reached the dynamite.

THE DUPONT POWDER MILLS at Laflin, Pa., were destroyed July 13 by an explosion, in which three men were killed and two injured.

AN EXPLOSION OF CREOSOTE occurred June 30 at New Orleans. A red hot rivet was dropped into a 5,000-barrel tank of creosote, which exploded, causing a loss of $100,000.

THE TORPEDO BOAT DESTROYER "Macdonough" was delivered to the Government by the Fore River Ship & Engine Co. of Quincy, Mass., on July 3, and was taken to the Charlestown Navy Yard. She is a sister to the "Lawrence," delivered by the same yard a few weeks ago and now at the Newport torpedo station. The "Macdonough" is 246 ft. long, 22 ft., 3 ins. wide and 14 ft. deep, with a displacement of about 400 tons. Her two engines have 8,400 combined horse-power, and with unselected steam coal carried her well beyond 26 knots at her endurance test, making 332.7 revolutions, against 331 required.

PROGRESS ON THE SIMPLON TUNNEL during the month of May, 1903, is reported as follows: Advance of south heading, 489 ft.; north heading, 571 ft.; a total of 1,060 ft. The entire penetration of the headings to the end of May was, therefore, 21,782 ft. in the south end, and 30,235 ft. in the north end of the tunnel, an aggregate penetration of 52,017 ft. The rock encountered by the south heading during May was mica schist, with some veins of gneiss, while the north heading advanced through mica schist and calcareous gneiss. An average working force of 3,532 men was employed, of whom all but 957 worked in the tunnel.

THE NEW ELECTRIC MOTOR CARS of the Central London Ry. (underground), which take the place of the separate electric locomotives formerly used in the operation of the line, have a motor truck at one end only. The cab covers the entire length over this truck, behind which the steel underframe is dropped low to receive the body of the car. The cab is 9½ ft. long and 6½ ft. wide over the frames, while the width over axle boxes is 7 ft. 2½ ins. The wheels are 34 ins. diameter, with a wheelbase of 6 ft. With a current of 200 amperes at 500 volts, each motor is designed to give a tractive effort of 2,500 lbs. at the rail, and a speed of not less than 18 miles an hour. At 80 amperes and 500 volts the tractive effort is 650 lbs. and the speed 27 miles an hour. The gear ratio is 1 to 4, with cut steel gears. There is a motor-driven air compressor, with a capacity of 35 cu. ft. of free air per minute compressed to 90 lbs. pressure in a main reservoir of 8 cu. ft. capacity. When the pressure reaches 90 lbs. the motor is automatically cut out, but the circuit is closed again when the pressure has fallen to 80 lbs. The Westinghouse quick-acting brake is used, and a hand brake is also fitted. There are 22 lamps on the car: 14 in the passenger compartment, one over the rear platform, three for the instruments in the cab, one in the switch compartment, and two in the headlight. The two latter of 32 c. p. and the others are of 16 c. p.

WIND PRESSURES sufficient to overturn a train caused an accident on the Leven viaduct of the Midland Ry., England, near Ulverstone, on Feb. 27, and "The Engineer" (London) publishes records of wind velocities in that vicinity at the time of the accident. The wind had a clear sweep from the sea up Morecambe Bay, 11 miles wide, to the viaduct, a distance of 9 miles. On each side of the mouth of the bay are anemometer stations. At Heysham, on the south side, the wind was from the southwest, with a velocity of about 55 miles per hour at the time of the accident, equivalent to a pressure of 30¼ lbs. per sq. ft. At Barrow, on the north side, however, the wind was from west to west-southwest, with a velocity of about 100 miles. Allowing for a decrease in velocity inland, the velocity at the viaduct would have been 70 to 80 miles an hour, increased to perhaps 90 or 100 miles during the severe gusts. It is considered certain that the higher velocity wind reached the viaduct and caused the accident.

THE NORTHWESTERN AVE. SEWER AT INDIANAPOLIS, IND.*

By W. Buehler.†

In March, 1901, a petition was handed to the Board of Public Works, of Indianapolis, for a sewer in Northwestern Ave., from Fall Creek to 32d St. (See Fig. 1.) The sewer has a total length of about 10,825 ft. With the exception of the overflow in 24th St., it is a combined sewer, the ordinary flow of sewage being intercepted where the main sewer turns into 24th St., and passing on down Northwestern Ave. to the branch of the White River intercepting sewer at 23d St. This branch governed the grade of the new sewer. From the very first it was seen that it would be impossible, owing to the low ground in 24th St., and along Northwestern Ave. from 24th to Roach St., to get depth enough for any ordinary sewer, large enough to drain the district. After careful consideration, the type of construction described below was decided upon.

Briefly described, the 5 × 10-ft. sewer, Fig. 2, which is the overflow section, extends from Fall Creek north to 24th St., then east to Northwestern Ave., at which point is constructed the dam previously mentioned and shown in Fig. 3. From here the 12 ft. 6-in. × 3 ft. 6-in. steel and concrete section, Fig. 4, is continued to Chicago St., where it is reduced to 11 ft. 6 ins. × 3 ft. 6 ins. (steel and concrete construction), which continues to Roach St., where it is reduced again to 8 ft. 4 ins. × 4 ft. 2 ins., concrete and brick construction, as also shown in Fig. 4. This continues to 28th St., at which point there is a bellmouth junction for the 28th St. branch. This bellmouth is shown in Fig. 5.

CONSTRUCTION.

The first step taken in the construction of the sewer was the driving of the piles to support that section of the sewer in the old creek bed in 24th St. About the same time the ground was broken on both sides of the old bed in 24th St., and also in Northwestern Ave., at 28th St.

At 24th St. the ground was taken out by means of scrapers, and as a good quality of gravel was found directly under the surface, it was all scraped to one side and immediately screened. As soon as the grade was reached, the forms were built and the concrete put in place. The sides and bottom were constructed first, the concrete being made from the gravel taken out of the trench, and mixed by a Smith machine. It was carried to the forms in wheelbarrows, and each batch (which consisted of six barrow loads) was thoroughly tamped in place. As soon as the concrete had set sufficiently the forms were removed and the brickwork commenced. The overflow sewer was rapidly completed.

At the same time that work was being done on the overflow, the section north of 28th St. was pushed as rapidly as possible, the sand and gravel being removed from the trench by means of a Potter trench machine. Work was also being done on the steel and concrete section of the sewer, which was begun at 24th St. The junction between it and the overflow (including the dam) was left to be finished some time later. At the completion of the work in the section north of 28th St. the trench machine was moved to 28th St., and the sewer running east in that street was commenced. The bellmouth junction in 28th St. was left to be finished as soon as that section of the sewer in Northwestern Ave. north of 24th St. was brought to that point.

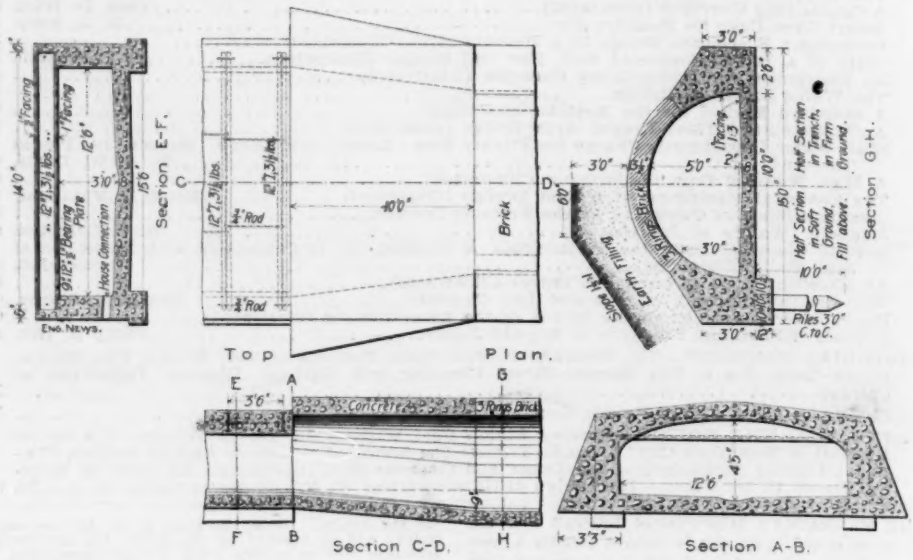


FIG. 2. OVERFLOW SECTION AND JUNCTION WITH MAIN SEWER AT NORTHWESTERN AVE. AND 24TH ST.

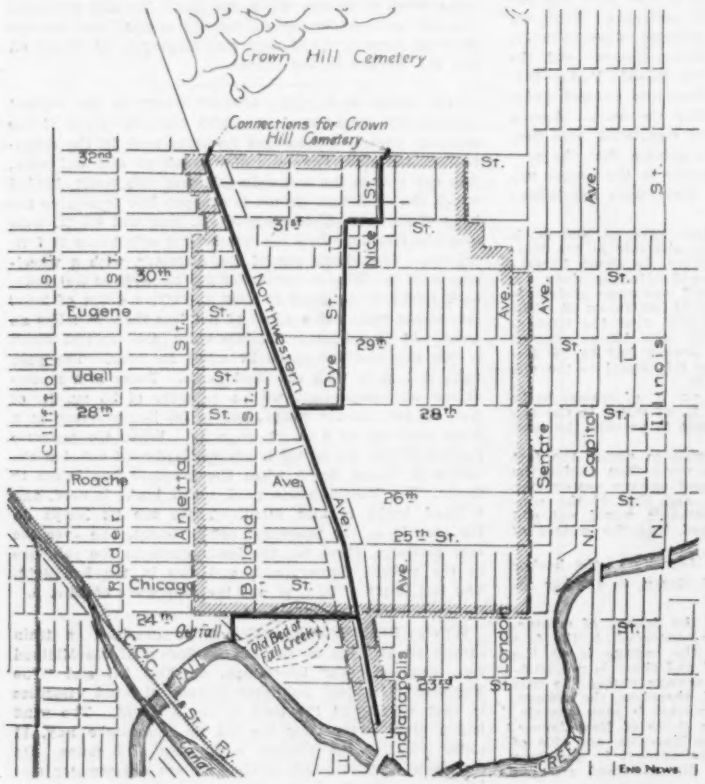


FIG. 1. MAP OF THE NORTHWESTERN AVE. SEWER AND ITS DRAINAGE DISTRICT, INDIANAPOLIS, IND.

*Condensed from a paper read at the annual meeting of the Indiana Engineering Society.

†Assistant City Engineer, Indianapolis, Ind.

SPECIAL SPECIFICATIONS.

The proportions for all concrete, except that used in forming the arch for roof of the sewer, shall be 1 part cement, 2½ parts sand, and 5 parts broken stone or gravel, by measure. For concrete used in the arch or roof, the proportions will be 1 part cement to 2 parts sand and 4 parts broken stone or gravel.

A natural mixture of sand and gravel may be used in place of the above if it is uniform in quality and contains no dirt or other foreign matter, or stones larger than above specified. In this case, the proportions for all concrete except that used in the arch or roof of the sewer shall be 1 part cement to 6 parts sand and gravel; and for that used in the arch or roof, the proportions will be 1 part cement to 4 parts sand and gravel.

The concrete floor, side walls and roof on the interior of the sewer, shall have a 1-in. mortar facing composed of 1 part Portland cement and 2½ parts sand. The exterior of the roof concrete shall have a ½-in. facing of the same. The mortar facing must in every case be built with the concrete and no plane of demarcation of plastering will be allowed.

On 24th St., in the old bed of Fall Creek, the sewer will be supported on timber piling as shown by the drawings. The piles shall be straight, sound, oak sticks not less than 10 ins. in diameter at the large end.

After the sewer has been constructed in 24th St., it shall be filled over and around the same, with earth or gravel. Sufficient material shall be used to thoroughly protect the sewer.

THE OVERFLOW SECTION.

The drawings of this are given in Fig. 2, and Fig. 6 is a view showing the work in progress. The forms for this section were built of pine in 16-ft. lengths and in three parts, the side walls, the cross-bracing and the back walls. The side walls were made of 1½-in. dressed boards; the cross-bracing of 2-in. stuff, and the back walls of 1-in. stuff. There were five cross-braces used to each length of form. In setting up the forms, the side walls were set on 8 × 8-in. blocks, placed approximately to grade. This allowed for the bottom and the side walls being constructed at the same time, and assured a perfect bond between them. The concrete was made of Gibraltar Portland cement, in proportions of 1 to 2½ to 5. The natural mixture being taken out of the trench containing too much sand, was passed through a ⅜-in. screen and mixed in the above proportions.

To get a perfect bond between the concrete and the 1-in. facing that was to be put on, a 1-in. board was placed against the form, which as the concrete reached the top, would be pried away from the form by means of a shovel or bar, and the facing (which was used rather wet) worked in. The forms were left in from five to seven days and then the centering for the brickwork was built on the bottom of the finished sewer.

STEEL AND CONCRETE CONSTRUCTION.

As before stated, the grade of the intercepting sewer in 23d St. governed the grade of this section. This, with the low ground along Northwestern Ave., between 24th and Roach Sts., necessitated the slight grade of 0.1%, and even then the top of the sewer came within 15 ins. to 24 ins. of the top of the ground. Taking into consideration the fact that the street railway track is to be laid over one end of the sewer, as shown in Fig. 7, it can readily be seen that the I-beams used necessarily had to be heavy, especially as some of the cars that will run over the track will be heavy interurban cars as soon as the Lafayette line is completed. The loading used in figuring the weights of the beams was a Union Traction Ry. car, weighing 31 tons light, plus 100 passengers at 150 lbs. each, 35,000 lbs. of the car to come on the front truck.

The forms were made of pine in 16-ft. lengths, the cross-bracing, side walls and back walls made of the same weight of material as that used in the construction of the overflow. The form for the back wall was used only in such places as would not permit of the natural wall of the cut being used as the back.

The forms for the top were made of 2-in. material, in 16-ft. lengths, supported by four supports spaced about 5 ft. apart. These supports were placed directly on the finished bottom. The bot-

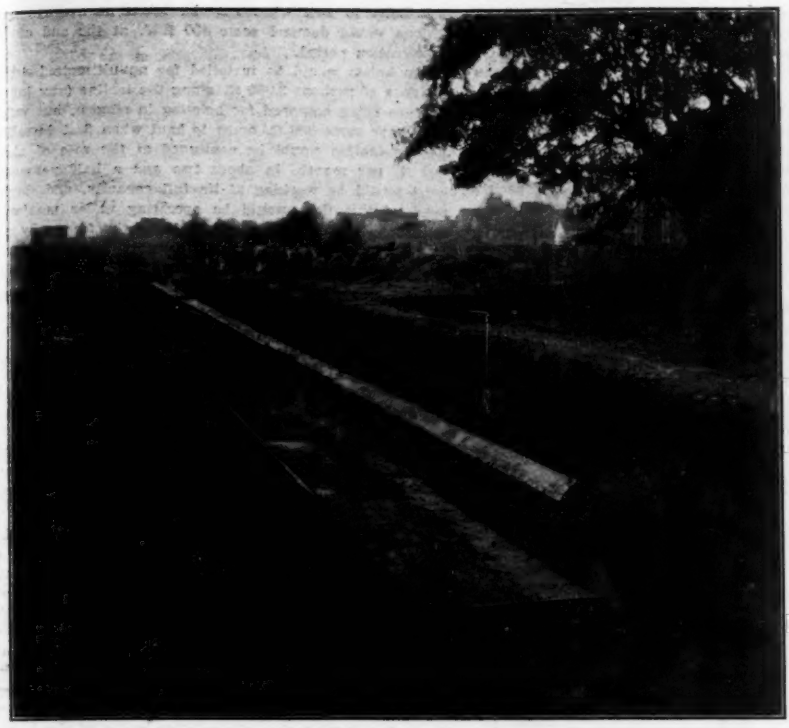


FIG. 6. CONSTRUCTION OF CONCRETE FLOOR AND SIDES OF BRICK AND CONCRETE OVERFLOW SEWER.
(The work is shown ready for the centering for the brick arch.)

FIG. 8. FORMS AND I-BEAMS IN PLACE FOR ROOF OF CONCRETE AND STEEL RECTANGULAR SEWER.
(The I-beams rest on bearing plates on the concrete side walls.)

tom was carried on far enough in advance of the top to allow this. The forms for the sides and bottom were placed in a similar manner to those of the overflow. The concrete and facing were placed in the same way, with the exception that in the construction of the bottom a depression 2 ft.

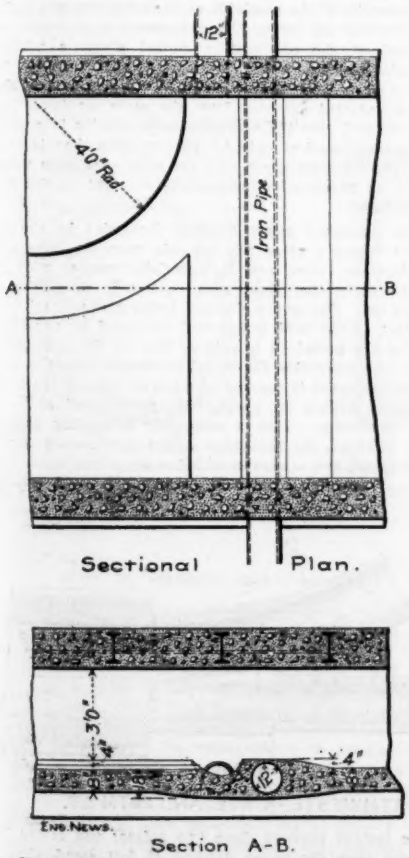


Fig. 3. Dam in Combined Sewer for Diverting Sewage Flow into Separate Sewer.

wide and 2 ins. deep at the center was put in to concentrate the sewage during the time of minimum flow.

The 12-in. I-beams, 31½ lbs. per ft., and 14 ft. long, were then put in. They were spaced 3 ft. 6 ins. c. to c., resting on a ½-in. bearing plate, 9 × 12 ins., placed on the top of the side walls. They were first put on 2-in. blocks on top of the form, the bearing plates were then set in neat cement mortar, and while the mortar was still green the beams were let down and forced to within 1½ ins. of the top of the form. A galvanized wire screen, 3 meshes to the inch, cut in 10-in. strips the length of the beam, was then put around the bottom flange, to hold the facing in place against the smooth surface of the beam. While this facing was still green, the concrete was put in level with the top of the beam, and covered by a ½-in. facing. Fig. 8 shows the form and I-beams in place on a curve, ready for the concrete.

SPECIAL CONSTRUCTION.

In connection with the steel and concrete section there are five details of special construction: (1) the manholes; (2) the construction over the 3-ft. laterals; (3) the junction at 24th St.; (4) the junction at Roach St.; and (5) the dam which crosses the junction at 24th St.

MANHOLES.—The sewer was so located with reference to the street car track that the manhole covers would be in the center of the track and the sides of the manhole flush with the wall of the sewer. This is shown in Fig. 7, and Fig. 9 shows the details of construction. The framing on which was built the brickwork of the manhole consisted of a 10½ × ⅜-in. plate, bent to a complete circle of 1 ft. 2 ins. radius and stiffened by four angles 5-16 × 3 × 2½ ins. The whole structure was wrapped with a galvanized wire screen and imbedded in concrete. The diameter of the bent plate was such that the city's standard manhole cover could be used.

LATERALS.—At 25th, 26th and 27th Sts., 3-ft. circular laterals entered the sewer. As the side walls at these points were only 3 ft. 6 ins. high, a special construction had to be resorted to. A 12-in. I-beam, 4 ft. 9 ins. long, was set lengthwise on the wall right over the place where the lateral would enter, and the 12-in. I-beam running across the sewer was framed into this. Fig. 10 shows the lateral connections at 25th and 26th Sts.

JUNCTION AT 24TH ST.—This was constructed completely of concrete. The bottom and sides were built in the same way as the two adjoining sections, the side walls of the junction tapering from 2 ft. 6 ins. (the thickness of the side walls of the overflow section) to 1 ft. 3 ins. (the thickness of the side walls of the steel and concrete section). The distance face to face of the walls increased from 10 ft. to 12 ft. 6 ins. The concrete for the top was placed on a timber centering built to conform with the top of the junction, which changed gradually from a segmental circular arch of 5 ft. radius to a segmental elliptical arch of 12 ft. 6 ins. span and 2 ft. rise. The junction was made in 10 ft. There was also a drop of 9 ins. in the grade at this point, all of which came in the junction.

DAM.—The dam which was constructed in the sewer at this point was made entirely of concrete, as already shown in Fig. 3, the top being 4 ins. above the flow line of the sewer and 3 ft. across. It ran diagonally across the sewer in the 12 ft. 6-in. section close to the junction. The center of the 12-in. intercepter was placed 9 ins. from the face of the dam, with its flow line 14 ins. below the top, the 2-in. depression in the sewer being gradually deepened to this depth. A 12-in. iron pipe crosses the sewer directly under the dam; the center line of it was 2 ins. higher than that of the intercepter. This pipe carries the sewage flow to a separate sewer. The concrete in the bottom at this point was lowered enough to completely cover this pipe to prevent any corrosion.

JUNCTION AT ROACH ST.—In this junction, the 11 ft. 6-in. rectangular section is changed in 15 ft. to an 8 ft. 4-in. semicircular section. The side walls of 11 ft. 6 ins. tapering to the 8 ft. 4 ins. The top is raised on a gradual incline and the bottom kept to the grade of the 8 ft. 4 ins. The top was constructed similar to that of the 11 ft. 6-in. section.

SOME NOTES ON CERTAIN UNDERGROUND HOISTING PROBLEMS ON THE WITWATERSRAND.*

By A. W. K. Pierce,† Assoc. M. Am. Inst. E. E.

Gold mining on the Witwatersrand consists chiefly of the exploitation of the "Main Reef Series," a bed of gold-bearing conglomerate with an east and west strike, and dipping to the south at a fairly uniform angle in the neighborhood of 30°. For an extent along the strike of some 50 miles this reef series has been proved to be gold-bearing, and the continuity of the deposit has been proved by actual mining and by bore holes to such a depth on the dip, that mining ground is held at a high valuation even where the ultimate mining depth cannot be less than 8,000 ft. (vertical) or even more.‡

*A paper presented at the 20th annual convention of the American Institute of Electrical Engineers, Niagara Falls, N. Y., July 3, 1903.

†Consulting Engineer, Consol. Gold Fields of South Africa, Germiston, Transvaal.

‡See "An Estimate of the Gold Production and Life of the Main Reef Series, Witwatersrand, Down to 6,000 Feet." By Messrs. T. H. Leggett and F. H. Hatch. Transactions Institution of Mining and Metallurgy. Vol. X (1901-2).

The permanence and the uniformity of average dimensions and values of these gold-bearing conglomerates make the mining in this area partake more of the nature of an industrial enterprise than is usually considered appropriate to mining ventures, which justifies the great attention that is paid to the engineering features of the work, and the large sums spent on the necessary mechanical equipment for the rapid and economical mining and reduction of the ore on a large scale.

According to the Transvaal mining laws, only the ore body contained vertically beneath the surface claim area can be mined by any given mining company. In order to exploit the ore contents of a mine situated at a distance from the outcrop, it is the practice, therefore, to sink one or more vertical shafts near the northern boundary of the property, to the intersection of the reef. This is then opened up by horizontal drives in the direction of the strike, and inclines along the dip; the principal inclines, in the most cases, being extensions of the vertical shafts. The ground thus opened up is stoped as required for the reduction plant, the development being kept ahead of these requirements sufficiently to permit of keeping up average values of the output, in spite of local variations in the gold contents of the ore body; and this process is continued until the payable ore body in the claim area is exhausted.

Where these vertical shafts are of moderate depth, the hoisting in both vertical and incline shafts is done by steam winders located on the surface, the vertical shaft being turned through a circular or parabolic curve into the incline. At greater depths separate winders are used for the two sections (vertical and incline), but both located on the surface, the ropes for the incline hoists being led down the vertical compartments near one side, and suitably guided into the incline, the ore being transfered from one hoist to the other of the intersection of the two shafts.

Further extensions of the "Deep Levels" involve vertical shafts of 3,500 to 5,000 ft. to the intersection of the reef, with inclines from this point of 3,000 to 6,000 or more ft. in length; and in planning the development and equipment of these new properties it is evident that present hoisting methods must be considerably modified. A recent paper by M. H. C. Behr* very ably presented the various features of this hoisting problem, and in this paper and the ensuing discussion will be found a very complete treatment of the matter from several points of view.

The concensus of opinion amongst South African engineers seems to be that these "Deep Levels" will be operated on an even larger scale than has obtained previously, the output planned for being 2,000 to 3,000 tons per mine per day; and that the greatest attention will be given to the economies that may be effected in working costs by the use of machinery, and in economic methods of operating the same. The advantages of electric power distribution are well recognized, and this method will be extensively adopted for what may be termed auxiliary power requirements. This is not an inconsiderable amount; it may be from 500 to 1,500 KW. per mine, for surface requirements only.

The underground workings will be arranged with a view to rapid removal of the ore contents, and it is probable that several incline shafts, operated by independent winding engines located underground, will be used to feed each single large vertical shaft, the latter having several pairs of hoisting compartments, and possibly arranged for hoisting in stages. In any case, there will exist within the next few years a demand for hoisting engines of considerable size, to be located underground in a dozen or more of the "Deep Level" mines, and with from three to six hoists per mine.

As a number of these mines in the same neighborhood are controlled by the same financial interests, and will require their equipment at approximately the same period of time, it is perfectly feasible to supply such a group of mines with electric power from a single central power station; and if the power required is to include that for the underground hoists it may be of interest to mention in a general way some of the conditions of the problem thus presented, and some of the conclusions that have been drawn therefrom.

In the first place: The principal power requirements of the mine equipment and the area over which the power is to be distributed (this being several square miles, with an added distance of in the neighborhood of a mile for each transmission to any underground installation) practically determine the use of a polyphase system at a moderately high voltage, especially as the ratio of copper cost to power cost is about twice as great in the Transvaal as in most countries, thus rendering the use of small line losses desirable.

However, I think it is generally conceded that for hoisting work on the scale indicated direct current motors are more suitable than alternating; their use in this instance, therefore, involves apparatus for converting the polyphase supply into direct current.

In spite of the losses and extra first cost involved in this conversion, it presents many important advantages in connection with the control system and the central station supply, as I shall endeavor to show.

First, however, as to the nature of the power demand of such hoists as will be required.

The hoists will be of the double drum type, with clutches for each drum; and while they will be operated normally in balance, they must be capable of bringing up a fully loaded skip from the bottom with the other drum unclutched, i. e., without the balancing effect of the empty skip and its rope.

While for each particular installation the size of the hoists must be separately determined to suit varying local requirements with regard to degree of inclination, load to be hoisted, length of haul, etc., the following may be taken as representing the approximate limits, between which particular values will be taken to suit conditions:

Load of ore 5,500 to 8,000 lbs. Hoisting speed 2,000 to 3,000 ft. per minute. Degree of inclination 25° to 45°.

A hoist to suit what may be taken as average conditions would demand some 400 KW. at the end of the acceleration period.

The hoists would be installed for an ultimate hoisting distance of perhaps 3,500 ft. along the incline (any longer inclines being equipped for hoisting in stages), but would only have some 300 ft. or so to haul when first installed. As the incline would be continued at the rate of about 100 ft. per month, in about two and a half years the hoist would be working at its full capacity, and during most of this time would be operating under maximum conditions of load, the variables being the frequency and the duration of the trips, and possibly maximum speed if the control scheme be suitable.

Considering an individual normal trip, it is evident that the maximum torque required from the hoist motor occurs at the time of starting the loaded skip on its upward journey; for the acceleration is a maximum, the moment and friction of the ascending rope is also a maximum, and the balancing effect of the rope on the descending skip is a minimum. On completion of the acceleration period the effective torque still continues to decrease during the remainder of the trip, by reason of the winding up of the rope on the ascending skip and the corresponding unwinding of the rope on the descending skip.

The design of the hoist should not be such that this torque would become negative before the completion of the trip, as this would require the application of the brakes at this point to destroy all the inertia of the moving masses; it is obvious that this inertia should be utilized as far as possible to complete the last part of the trip, the brakes only being used to effect the final landing of the skip.

While similar in kind, the mechanics of this problem are quite different in degree from that of operating elevated railway trains, the relative value of the kinetic energy of the moving masses and the work done against friction and gravity being quite different in the two cases. Thus with the limitations to hoisting speed imposed by practice, a considerable part of the trip must be made with power after the acceleration is completed, and the "coasting" period only exists for the few seconds required to bring the moving masses to a standstill.

In considering the demand on the generating plant occasioned by such a trip, the form of the acceleration curve to be adopted is of importance.

Obviously, if the available acceleration moment is fixed, maintaining the moment at a constant (maximum) value throughout the acceleration period gives a minimum duration of that period; but, as pointed out by Mr. Behr in the paper previously referred to, an acceleration showing a constant decrease from the same maximum value to zero will give the same ultimate velocity in a certain space, and requires only 5% greater time than is needed to cover the same space with the same maximum velocity using an acceleration maintained constant at the maximum value.

The maximum power actually demanded by the hoist motor during a given trip depends upon the form of the acceleration curve, and is materially greater with constant acceleration than with constantly decreasing acceleration. The power demand being proportional to the product of the total torque and the speed at any instant during the period, it is evident that as the constant acceleration maintains the initial maximum torque until the maximum speed is reached, the power demand continually increases during the acceleration period, and at its end is a maximum. With a constantly decreasing acceleration, however, the maximum torque only occurs at minimum speed, and at maximum speed the acceleration torque is zero; therefore the maximum power demand occurs at some instant between these two points, and is but little greater than the power demand at full speed after acceleration ceases, which is that required for overcoming the effects of friction and gravity only. The actual amount of the work done is of course practically the same in either case.

It is evident that the constantly decreasing acceleration is much more favorable to the central station, in that it gives a better load factor, less fluctuation in the power demand, and requires less investment in generating plant, provided the method of control is such that the power de-

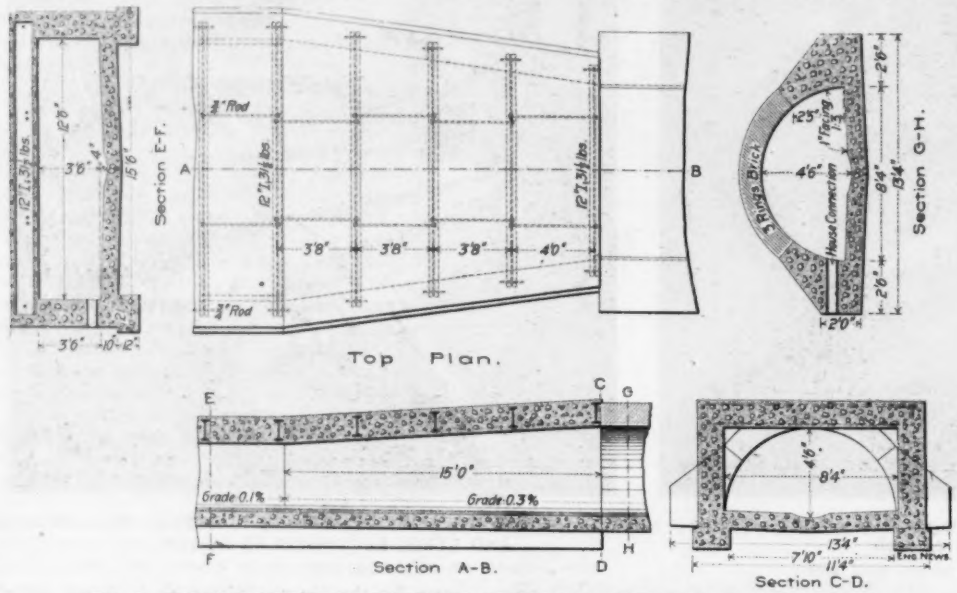


FIG. 4. REDUCING SECTION OF SEWER AT NORTHWESTERN AVE. AND ROACH ST.

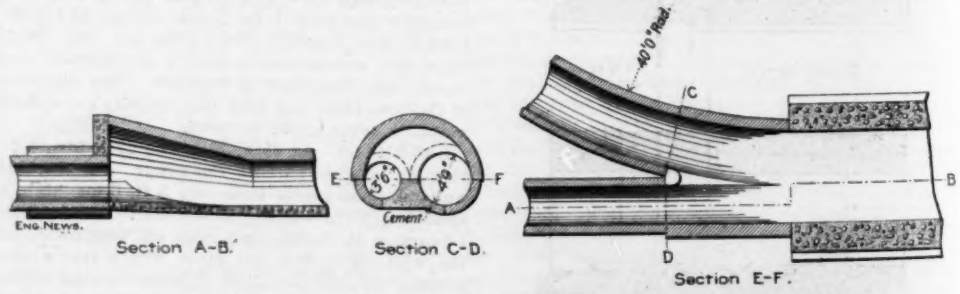


FIG. 5. BELLMOUTH JUNCTION AT NORTHWESTERN AVE. AND 28TH ST.

*"Winding Plants for Great Depths." Read before the Institution of Mining and Metallurgy in London, May 15, 1902, and before the South African Association of Engineers, in Johannesburg, August, 1902.

mand from the source of power is proportional to the actual power demanded by the hoist motor.

The control system is thus the important factor in attempting to realize these advantages, as the decreasing acceleration is more normal for electric motor operation than the other; and for this purpose I know of no other method of control that is workable than some form of "voltage control" or "Leonard" system. Involving as this does some separate dynamo machinery for each motor installation, the advantage in control method previously referred to in connection with the conversion from the assumed polyphase distribution to direct current becomes apparent, in that this converter machinery may be utilized in one of a number of obvious ways to furnish the "voltage control" of the hoists.

This machinery also helps out the central station in another way, in that the inertia of its moving parts further smooths out the peaks in the demand curve during each trip; this effect may even be increased by the deliberate addition of inertia to the converter set in the form of a flywheel, as has been proposed by several Continental engineers that have investigated this problem.*

The "voltage control" also possesses incidental advantages. The necessary slow speed inspection trips in the shaft may be readily made; during the first year or so of operation, the controller can readily be provided with adjustable stops limiting the maximum speed (consequently the maximum power demand) to whatever amount will give the desired output, and thus again improve the central station load factor; the controller need not handle the very considerable main current of the motors, thus eliminating a source of considerable first cost, and also maintenance cost; given positions of the controller handle result in nearly enough given speeds, irrespective of loads. These characteristics are so well known that it is perhaps superfluous to mention them here; I only wish to indicate that in almost every particular this problem is best met by this control scheme, which favorably affects both the source of the power and its utilization.

I have only stated the conditions in a general way, without particular data as to details, for much of the data has not yet been definitely determined, and would necessarily vary with the different local conditions of different mines. I trust that in spite of this, I have justified to some extent the conclusion that for this important hoisting work, the best method is the combination of polyphase transmission, conversion to direct currents separately for each hoist unit, and the utilization of the conversion machinery for obtaining suitable control of the hoisting operations by the "voltage control" or "Leonard" system.

*See "Electric Winding Problems." By H Snengel, Transactions South African Association of Engineers, 1902.

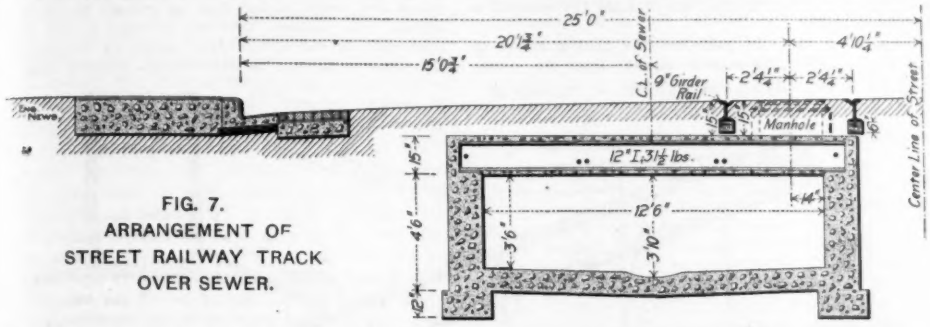


FIG. 7.
ARRANGEMENT OF
STREET RAILWAY TRACK
OVER SEWER.

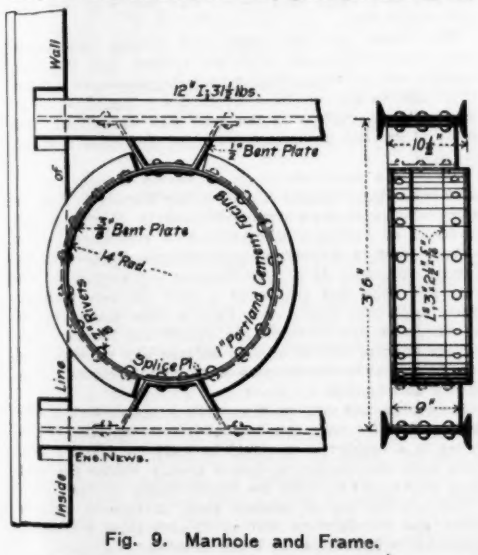


Fig. 9. Manhole and Frame.

THE HEPPNER DISASTER.*

By John T. Whistler,† Assoc. M. Am. Soc. C. E.

On June 14 there occurred at Heppner, Ore., one of those storms so peculiar to arid regions and generally referred to as "cloud-bursts." A knowledge of the amount of precipitation and the area covered, seemed to be of especial interest in the consideration of irrigation projects now being investigated by us in this State, the lands for one of which lies about 40 miles from Heppner.

I have made an investigation to determine more or less approximately the facts as to run-off and area of storm, and submit herewith the results of this investigation, together with other facts which I have thought may be of general interest.

Heppner, the county seat of Morrow Co., and a town formerly of about 1,400 people, is on Willow Creek, 45 miles south of the Columbia River, into which Willow Creek empties. The mouth of Willow Creek is about 220 ft. above sea level. The elevation of Heppner at the railroad station is 1,920 ft. above sea level. Willow Creek valley is a fertile strip of cultivated land, 500 to 1,500 ft. wide. On both sides lie rocky and barren hills, which carry off any rainfall very rapidly. Hinton Creek empties into Willow Creek at the lower end of the town. About a mile above Heppner, Balm Creek empties into Willow Creek. It was down this valley that probably 90% of the flood came.

The most authentic account of the storm, from residents of the valley and other eyewitnesses, agree that it moved down the valley of Balm Creek, or the side of the divide next to it, and had entirely passed away in less than a half-hour. Accounts agree that it extended from the foot of the Blue Mountains, 8 to 10 miles distant, to and a short distance across Willow Creek valley, covering a strip, variously said to be from 2 to 4 miles wide. The rainfall was preceded by a phenomenal fall of hail.

High water marks are everywhere plainly visible. Two cross-sections were taken in Balm Creek valley, 2,200 ft. apart, and the fall between measured. Three cross-sections were also taken at Heppner and two at Lexington, nine miles below and 500 ft. lower in elevation. The time of beginning, crest, and end of flood was obtained from two or more people at the different towns along the valley. Near the mouth of Willow Creek, for a distance of about 1,000 ft., the creek flows through a regular channel, constructed by the Oregon Railroad & Navigation Co., and riprapped on one side to protect the embankment. A mean cross-section and fall of channel were taken here.

As the flood advanced down the valley, the first waters and the crest of the flood were almost simultaneous as far as Lexington, nine miles below. At Ione, eight miles farther down, they were a half-hour apart, and at Cecils, 14 miles above the mouth, first waters and highest water were several hours apart.

At Heppner the creek is said to have been almost normal an hour and a-half after the crest of flood, which was almost if not simultaneous with first water at 5.30 p. m. At Lexington there was possibly a difference of 15 minutes between first water and crest, the first water reaching the town at about 7.00 p. m. and lasting three hours before the creek was near normal. At Ione, 17 miles below Heppner, there was a half-hour between first water and crest, the first water reaching that place about 10.00 p. m., and lasting about seven hours. It was 20 hours before highest water reached the mouth of the creek at Heppner Junction, the true flood crest having disappeared probably 20 miles above.

Each of the two flood cross-sections taken on Balm Creek give an area of approximately 1,700 sq. ft. The slope of the flood between was 0.025, or about 130 ft. to the mile. Taking the wet perimeter as 260 ft., "n" as 0.040, and using formula $v = c\sqrt{rs}$, the discharge amounts to 36,000 cu. ft. per sec. It is probable from the evidence at Heppner, a mile below, that the entire duration of flood at this point was not over an hour. It is even said by some that in 20 minutes it was possible to go about the town by some wading. From this and the account by two witnesses to the Balm Creek waters, I am confident that a maximum discharge for 20 minutes would more than equal the total discharge of Balm Creek from beginning to end of flood. On this assumption the total discharge amounts to about 1,000 acre-feet.

The total discharge at Heppner, calculated in the same way, that is, assuming the total discharge to be equal to the maximum discharge for one-third total time of flood, and estimating the velocity at six miles per hour, as shown by the time required to reach Lexington, with an area of cross-section of nearly 3,000 sq. ft., amounts to 1,100 acre-feet.

At Lexington, with a cross-section of 1,600 sq. ft., and estimated velocity of four miles per hour, the discharge amounts to about 800 acre-feet; a loss of 300 acre-feet. This seems excessive, and it is probable that the cross-sections taken were somewhat narrower than the mean of the valley above, and the velocity resulting somewhat greater than four miles per hour.

At the straight, regular section near the mouth of Willow Creek, the high water cross-section has an area of 235 sq. ft.; the fall is 0.01, or 53 ft. to the mile; wet perimeter is 38 ft.; and the value of "n" is probably not more than 0.030. First water began at 10 a. m. and increased till 2 p. m. of the day following the flood at Heppner. From 2 until 3 p. m. the water was stationary, and from 3 p. m. till 5 a. m. decreased to near the normal. From this a total discharge is estimated of 300 acre-feet during this time. On June 18 the discharge was 10 sec.-ft. at Heppner and 35 sec.-ft. at the mouth of Willow Creek. Assuming that this difference of 25 sec.-ft. will be an average of the run-off below Heppner each day, and that the stream will have been reduced to normal at the end of a week, 350 acre-feet more will be added to the total discharge at the mouth, making 750 acre-feet.

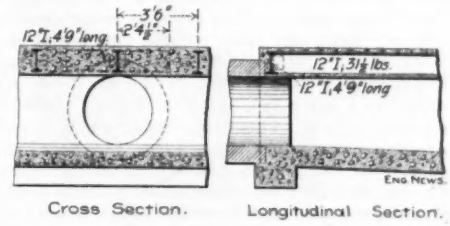


Fig. 10. Lateral Connections.
(I-beams of Main Sewer Framed against I-beam over Lateral.)

The increase at Heppner over Balm Creek is due to water brought down Willow Creek from above Balm Creek.

It seems extremely improbable that the area of rainfall was less than 20 square miles. Assuming that two-thirds of the precipitation reached Heppner, the total precipitation would be approximately 1,600 acre-feet, or 1½ ins. depth over an area of 20 square miles, the probable area of storm.

About one-third of the town is entirely destroyed, at a property loss in connection with that of the railroad and the comparatively small loss at Lexington, of about $250,000. The loss of life was very near 200, 170 bodies having been recovered on the 20th, and probably 50 still unaccounted for.

A curious fact, about which there is no doubt, is

*Copy of a report made to the U. S. Geological Survey, sent through the courtesy of Mr. F. H. Newell, M. Am. Soc. C. E., Chief Engineer, as the result of a request to Mr. Whistler from this journal for a report on the flood.

†U. S. Geological Survey, Pendleton, Ore.

that almost without exception, the bodies recovered have been nude or nearly so. The bodies have usually had on shoes, those of women frequently a corset, and those of men occasionally a collar.

The excessive amount of hail has been noted. Some of the hail stones are said to have measured 1¼ ins. in diameter. I was unable to find anyone who could himself verify this, but I measured the largest one of a random handful picked up five days after the storm. It was ⅝-in. by 7-16-in. and could easily have been ¾-in. diameter when it fell. A grim evidence of the amount of hail that fell is that, while most of the bodies being recovered on the fifth day were already badly decomposed, one was occasionally found almost perfectly preserved in a large drift of hail. Nearly all the hail was of clear ice, and unlike the usual hail stones, which are of a more opaque ice, was built up from a nucleus in successive layers.

The facts can in no way detract from or belittle the awfulness of the disaster, but it is of interest to note from the standpoint of an investigator, some of the unquestionably honest but just as unquestionably inconsistent and exaggerated statements made by those who were eye witnesses to the flood. The most conservative make the flood wave to have been 15 ft. high, and it has been reported in at least one reputable newspaper that it was 50 ft. high. Stories are repeated of people whose houses withstood the waters, and that they stood knee-deep in water on the second floor. The facts are that the water was in no place, outside the creek banks, over 6 ft. deep, and not over 5 ft. deep where any house stood. A gentleman in the Palace Hotel, one of the few brick buildings in the town, told me that all that saved the hotel was another building which swung around in front of it. I then asked him if he could show me the high water mark in the hotel. He admitted it did not get over the floor. The facts are that the water was barely a foot higher than the sidewalk at the entrance to the hotel.

Nearly all of the houses are of light frame construction, usually so built that the water could readily pass under them, at least more readily than into them, and there is little doubt that the houses were in most cases carried off like boats. An evidence of this is that of the hundred and more houses destroyed, there is but one piece of floor left in place. This had a cellar under it and the walls were fairly tight under the floor. It would seem likely that the light studding walls were knocked over by some other floating house and the floor held down by the sheer weight of water on it, the cellar not having had time to fill. There are instances of buildings, where the doors were left open, having withstood the waters while others next to them with closed doors were carried away.

Almost exactly fifteen years ago, a similar flood, possibly larger, came down Hinton Creek, which empties into Willow Creek at the lower end of the town. It did little damage then, for there were few buildings in its way. One or two lives were lost at Lexington, I believe. Twenty years ago, there was another heavy flood in Willow Creek, but of this I could get little information.

It has occurred to me that the great destructiveness of these so-called "cloud-bursts" is due more to the rugged character of the topography, and the almost utter absence of vegetation, than to the unusual rainfall. It appears to be true in this instance at least.

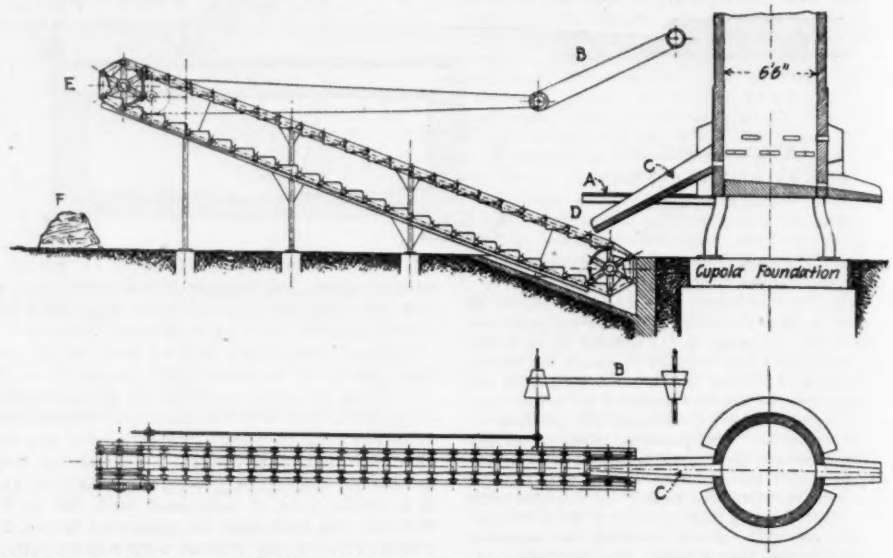
A NEW CUPOLA SLAG CONVEYOR.*

By Thos. D. West.†

In melting over 50 tons in cupolas of above 50 ins. inside the lining, there is more or less slag formed which must be taken from the cupola. Some founders allow this slag to accumulate below the slag spout, while others prefer to convey it some distance away. To do this the slag is often let run down a sand or iron runner as far as it will flow, and is then pulled with a hook the balance of the distance, while in a semi-molten state. Other founders allow the slag to run into small buggies which can be handled by one man. Again the slag may be let run into a car which is sent to the dump pile over regular track.

Believing that present methods could be improved, that money could be saved, besides doing away with disagreeable labor and aiding our continuous melting system, we decided to install a slag conveyor as shown in the accompanying illustration. This conveyor was gotten up by the C. O. Bartlett & Snow Co., of Cleveland, O. While it might be thought a rather elaborate and costly machine, nevertheless, as we are making radical changes in our methods of molding and casting, and are arranging for continuous melting of 20 to 22 tons per hour, or 480 to 528 tons daily for each week's steady run, when using hot metal from the blast furnace, we consider the machine a good investment.

In operating the slag conveyor, the belt B serves to start and stop it. The slag as it runs from the cupola



CUPOLA SLAG CONVEYOR AT THE WORKS OF THE THOMAS D. WEST FOUNDRY CO., SHARPSVILLE, PA.
The C. O. Bartlett & Snow Co., Cleveland, O., Builders.

flows down the spout C into the buckets D. The buckets convey the slag outside the shop, dropping it from them at E in solid form, into the car, cart, or pile F. The blocks, after they are sufficiently cool, may be broken up to pick out the iron which has been carried out with the slag. We have a method in view which contemplates gathering the iron before dropping the slag into the car. This will then save the labor connected with the slag question in any form.

When it is considered that slag may contain shot iron up to 4% as well as an oxide of iron equivalent to 15 to 20% of the metal, aside from what iron may run out over the slag hole, the importance of gathering all the metallic iron possible may be seen. Then again, as the amount of slag produced is about 30 to 50 lbs. per ton of iron melted, the slag made by our projected daily heats would be 19,000 to 25,000 lbs. From which the importance of the new slag conveying system we are installing may be readily judged.

*A paper presented at the Annual Convention of the American Foundrymen's Association at Milwaukee, Wis., June 9-11, 1903.
†Sharpsville, Pa.

RETORT OVEN COKE FOR FOUNDRY USE.*

By W. J. Keep.†

For 1½ years I have used retort oven coke exclusively, melting about 80 tons of iron per day. I have used coke made at three different plants, from Hamilton, O., Sharon, Pa., and Detroit. This coke was made from a great variety of coals from West Virginia and Pennsylvania. The coke used during 1902 was made in Detroit and was uniformly good. At times during January and February of this year we received some coke that was not satisfactory because the makers could not procure proper coal owing to transportation difficulties. When this condition changed the coke again became satisfactory and has continued to be so.

During these same early months of 1903 founders all over the country had trouble with coke made in beehive ovens. Founders in many parts of the country were obliged to take any kind of coke that they could get in order to keep running. During the past season most of the best deposits of coking coal in the Connellsville region have been purchased by the United States Steel Corporation and very little genuine Connellsville coke is available for general foundry use.

It is evident from a comparison of these two methods that coke made from the same coal will have a slightly higher percentage of fixed carbon and a slightly lower percentage of ash if made in a retort oven. The beehive oven is almost universally located at the mines from which it draws its supply of coal. Retort ovens are located in places that provide a market for the coke and gas.

To allow a continuous operation of a retort-oven plant the product must be sold as fast as made and to do this a uniform quality of foundry coke must be maintained. The quality of retort-oven coke depends upon the skill of the operator, upon the method of preparing the coal and more than all upon the quality of the coal used.

Our experience with retort-oven coke began during the fall of 1901. We were at first induced to try one car but when it arrived it was so black and in such small pieces and looked so bad that we refused to try it in a cupola. The makers, however, proposed to pay for all poor castings and under these conditions we used it in the same proportion as the Stonega beehive coke that we had been using previously. The iron was very hot and satisfactory.

We ordered five cars more, and ordered retort-oven coke from other ovens until we decided that the coke worked well in spite of its very bad appearance. From that time to this we have never had a pound of other coke. All through 1902 the coke was so uniform and satisfactory that we melted 9 lbs. of iron with 1 lb. of coke.

There is a great advantage in having a local retort-oven coke plant, because it enables the founder to keep in close touch with the source of his supply. Owing to the necessity of having a laboratory at a retort-oven plant to keep track of by-products, the coke manufacturer keeps himself informed as to the chemical composition of his coal and coke and can make a very uniform product. Such local coke plants will have a very large variety of customers and therefore can supply the founder with a very uniform quality of coke because any inferior coke can be sold to domestic users or to manufacturers where it will do no harm.

Experience has proven that with a retort-oven satisfactory foundry coke can be made from coal that when coked in a beehive oven would be unfit for foundry use. This helps the founder in that it greatly widens the field from which he can draw his future supply of fuel.

Retort-ovens are at present being erected in various cities, and I understand that an 80-oven plant will be in operation in Milwaukee in a few months.

*Extract from a paper presented at the annual meeting of the American Foundrymen's Association at Milwaukee, June 9.
†Detroit, Mich.

REPLACING A SUSPENSION BRIDGE BY A MASONRY ARCH.

A rather special piece of work was done recently in replacing a highway bridge crossing the Dordogne River at Argentat, in central France. The existing structure, a suspension bridge of 100 m. (328 ft.) span, erected in 1829, had in recent years become inadequate, so that it was considered necessary to replace it. The new bridge is a masonry arch bridge of three spans, each 32.64 m. (107 ft.) in the clear, and has the same longitudinal axis as the old bridge. The total length of the new bridge between end abutments is 108.85 m., which is 8.85 m. (29 ft.) greater than the distance between the old abutments. It was therefore possible to utilize only one of the existing abutments, while the other had to be cut away to make room for the new abutment and part of the

arch ring. This would seem to involve taking down the old bridge before beginning the new work; on the other hand, it was desired to interrupt traffic as little as possible, without going to the expense of a temporary bridge. A method was adopted therefore by which abutments and arches could be built without removing the old bridge, so that the interruption of traffic was confined to the time required for filling and finishing the bridge above the arches.

As already stated, the abutment on one side was made to serve directly as abutment for the new bridge; the preparation of this abutment and the construction of the two river piers was first completed. The other abutment and the arch ring were built in the manner illustrated by the accompanying cuts, Figs. 1 and 2. The pier below the masonry towers supporting the cable-saddles was reinforced on either side by a stone masonry buttress, as shown in Fig. 1. Just below the level of the roadway each of these buttresses carried a pair of plate girders, parallel to the bridge (A and B, Fig. 1), and upon these rested seven pairs of transverse I-beams which were successively drawn in as needles under the towers and transmitted the tower load to the outer buttressed portions of the pier. The middle part of the pier was then removed, to the lines shown in Fig. 1, and in this way sufficient clear space was obtained to construct the arch. The abutment is an extension of the old anchorage

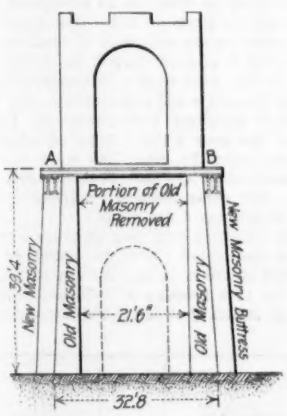


Fig. 1. Reinforced Pier Carrying Cable Towers.

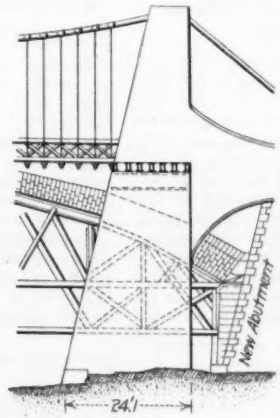


Fig. 2. Building New Arch Through Old Pier.

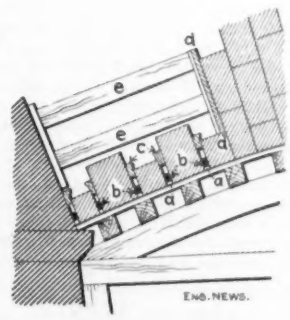


Fig. 3. Closure of Arch at Skewback.

abutment, and falls in the vaulted space between this abutment and the cable tower, and is shown in Fig. 2.

In building the arches the novel plan was adopted of making the final closure at the haunches instead of at the crown. The work was carried on as follows: The falsework being in position and properly lagged for the reception of the arch stones, the lower course of the arch ring was started by laying five rows of archstones adjoining the skewback (see Fig. 3). These five rows were laid dry, i. e., without mortar, and were held apart with wooden wedges so that the mortar might be rammed in subsequently. Along the lower edge of each joint a strip of wood, a, 3-16-in. thick by ⅝-in. high, was laid in the joint, and above this was a strip of sheet lead, b, which was also 3-16-in. thick (the width of the joint), but 1½ ins. wide. The upper edges of the blocks were held apart by wooden wedges, c. At the end of the fifth row a wooden partition d d was erected above the first course of stones, and was braced against the skewback by the struts, e. The arch ring beyond this partition was then laid complete. To close the arch between the skewback and the partition, d, cement mortar mixed quite dry was rammed as firmly as possible into the joints between the wedges, above the lead strip. As the filling proceeded the wedges were removed one by one. The first course being thus completed, the second and succeeding courses were laid in regular manner, the struts, e, and the wooden strips composing the partition, d, being taken out as the work progressed.

The centering was supported on sand boxes. The lowering of the centers after the arch masonry had set proceeded with great care; a man was stationed at each sandbox with a pail holding just enough sand to lower the support one millimeter. Levels taken on the arch showed no movement during the lowering, which proceeded by successive stages of one millimeter. After the centering had been removed, the ⅝-in. wooden strips in the joints were taken out and the joints pointed up with cement mortar, the lead strips being left in place. The entire work, which was carried out as above outlined by Mr. Charron, as engineer, was described in a recent issue of the "Annali della Societa degli Ingegnieri e degli Architetti Italiana," to which journal we are indebted for the sketches reproduced here.

TESTS OF A DIRECT-CONNECTED 8-FT. FAN AND ENGINE.*

By E. S. Farwell, M. Am. Soc. M. E.†

The promoters of the plenum system of heating and ventilating have obtained a very substantial footing among the paper mills, because of the moisture which it is necessary to remove from the rooms.

For instance, in a room containing two paper machines, each making 25 tons of paper in 24 hours, there is approximately 100 tons of water which must be disposed of before it condenses on the roof and trusses, and drips on the machines. Also, in pulp-grinder rooms, there is a large amount of steam generated which must be disposed of, or the fog in the room will be so dense as to impede the work. In most of the other rooms, however, it is merely a question of heating.

I found it rather difficult to obtain reliable data which would guide me to an intelligent selection of apparatus for any particular job. The builders of such apparatus very generously offered to work out the problem for me, but it is safe to say that in no case were any two proposals for the same size of apparatus. "A" would submit a proposition for a small fan, running at high speed, with small air-pipes and a small heater; "B" would offer a large fan to run at a slow speed, with large air-pipes and a large heater; "C" would perhaps offer a large fan, with large air-pipes and a small heater. If "A" were asked to furnish a large fan to run at a slower speed, he immediately proposed to enlarge the pipes proportional to the size of outlet of the fan, and to put in a much larger heater, notwithstanding the fact that the amount of heat to be delivered was the same. I wish to further acknowledge that under these perplexing difficulties the representatives of the different fan builders never hesitated to give me all the assistance which they apparently could, and I believe that I have finally succeeded in getting some points clear in my own mind.

The question seemed to me to embody three distinct problems: the air-pipe, the fan, and the heater. The first two are interdependent, or perhaps we had better say both depend upon the same assumed data. The starting-point of the problem or problems is the volume and temperature of air required. In a factory building, if it is merely a question of heat, a comparatively small volume of air at a high temperature is satisfactory. A change of air as low as 30 minutes is allowable where men do not stand at their work, and 20 or 25 minutes where they do. In the grinder-room the change must be as frequent as 15 or even 10 minutes, depending on the shape of the room, the location of the grinders, kind of roof, and other local conditions. In paper machine rooms, also depending on local conditions, the required change may be as often as 4 minutes.

It is very easy, with the published tables and data of tests, to determine the amount of heating surface required to heat the determined volume of air to the desired temperature. I never could see how the size of the fan need affect the amount of heating surface, provided the volume to be delivered and initial temperature of air were the same in both cases.

About 1-16 oz. pressure per square inch will be required to give the necessary velocity at the outlets of the distributing pipes. In addition to this there must be as much more pressure as is necessary to overcome the friction of the pipes, and herein I find is the principal difference in the practices of the different builders. Some always figure on 1 oz. pressure at the fan. Others figure ⅝ oz. Fifteen-sixteenths of an ounce seemed to me to be an excessive loss, and I usually call for ½ oz. at the fan, enlarging the pipe sufficiently to bring the friction loss well within that figure.

In reducing the pressure required of the fan, we have of course reduced the amount of work done by the fan; but against this saving in work, it is necessary to charge the interest and depreciation on the increased cost of fan and pipe.

In order to secure some data for my guidance, I made a series of experiments on the efficiency of a fan which I purpose to outline in this paper. The fan was a No. 100, according to the usual method of designating fan sizes, with a wheel 8 ft. in diameter, and 37 ins. wide at the periphery and with one side inlet. It is shown in Fig. 1. The fan is driven by a direct-connected steam engine, and discharges into a large chamber supplying air for combustion to the boilers. This chamber was left open to maintain atmospheric pressure except in a couple of tests. The opening of the fan into this chamber was tightly boarded up, and conical tubes or nozzles were fastened to circular openings in this board partition. The tubes had a taper of 3½° and, with the exception of the largest two sizes, were approximately three times the diameter in length. Six different sizes of tubes were tested, each at 8 or 10 different speeds, ranging from 50 to 250 revolutions per minute. In addition, a set of observations at each speed was made with no outlet whatever.

This I realize is not an ideal arrangement, but under the circumstances seemed to be all that the desired results would warrant. The volume of air delivered was determined by means of a pitot tube located at the extremity of the conical outlet. We attempted to use an anemometer, but some of the velocities were beyond the capacity of the instrument and it very soon proved to be unreliable. With the tapering outlet, it was apparently safe to assume that the air at the extremity had no static pressure, but that the entire potential energy had been converted into kinetic energy. The pressure in the fan chamber, as also the vacuum or suction at the inlet, was measured by a water column. We found the vacuum at inlet varied at the different points of the inlet as the gage was moved from the center toward the rim. A number of observations were made, and a point determined which gave us the mean of all the readings. The temperature of the air was taken, also the revolutions of the fan and indicator cards from the engine. This completed the list of observations taken during each test. Each test was run for half an hour under uniform conditions, and readings taken every five minutes. The average results of each test are given in the table. The volume of air discharged was computed from the velocity of air as given by the pitot tube, and tabulated in column marked "B." The velocities due to pressure in the chamber have been tabulated for comparison only, and are not used in the computation of results. The theoretical horse-power was computed from the volume, determined as above noted, and the total head as tabulated. The pressure, volume, indicated horse-power, and efficiency have also been plotted on the two sets of curves, Figs. 2 and 3. In Fig. 2 the abscissæ represent revolutions of fan, while each curve represents a certain size of outlet. In Fig. 3 these two items have been interchanged, and each pressure curve has a different zero line. It is the custom, I believe, to designate the size of fan outlet in terms of the diameter and peripheral width of fan-wheel, and this I have done in the present instance.

A number of interesting facts stand forth very clearly upon an examination of these curves. Theoretically, the volume should vary directly as the speed of the fan with a given size of outlet, the pressure as the square of the speed, and the horse-power as the cube of the speed. The curves show that the ratios are not quite those stated. Up to a certain point the volume curves are very nearly straight, but at the higher speeds they seem to fall off. This falling off was more marked with the large openings, as was also a large increase of the vacuum at inlet. This loss in the volume delivered is unquestionably due to the throttling of the air at inlet. The maximum efficiency of the combined unit was secured at 142 r. p. m., when the pressure was ¼ oz. (this, of course, only applies to the particular fan tested, and it is fair to presume that if the fan had had an inlet on each side, the throttling would have been less and the most efficient speed might have been higher).

The curves show very clearly, however that in the selection of fan we should choose large sizes running at

*A paper presented at the Saratoga meeting of the American Society of Mechanical Engineers.

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moderate speeds and developing a low pressure. We may apparently run the same fan at a lower speed and lower pressure so that it will deliver the same volume of air with a considerable saving in horse-power. This should be done, if in any case the conditions prevent the selection of a larger fan. As I will illustrate in a few moments, however, the larger fan will do the work more efficiently.

It has been frequently stated that up to a certain size of outlet, variously styled the "theoretical outlet," "square inches of blast," and "capacity of wheel," any change in size of outlet makes no change in the pressure, and a variation in volume and horse-power directly proportional to the size of the outlet, and that further enlarging results in a decided drop of pressure and falling off in the rate of increase of volume and horse-power.

The curves show that up to about .29 D. W. the pressure drops but slightly. The volume and indicated horse-power rise very nearly by straight lines, but beyond that point there is

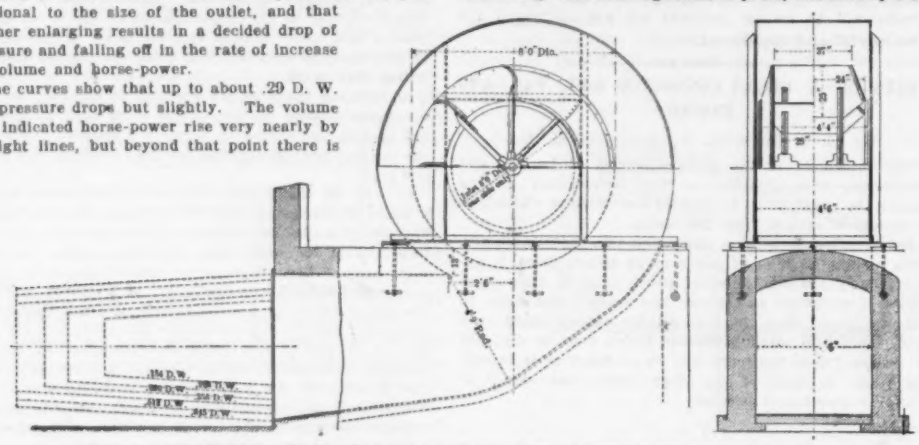


FIG. 1. SECTIONAL ELEVATIONS OF 8-FT. FAN AS ARRANGED FOR TEST.

a sudden and rapid change, the efficiency also reaching a maximum at this same point.

As being probably pertinent to this point, I wish to call attention to the fact that the area of the fan-table is equal to .2885 D. W. The efficiency curves show that but slight variations from this "theoretical outlet" should be permitted. Mr. Snow states that for general practice the square inches of blast is not far from $\frac{\text{D. W.}}{3}$. I understand that the usual width of periphery of an 8-ft. fan, as built by the makers of the fan tested, is 41 ins. instead of 37 ins. This would make the area of fan-blades approximately .32 D. W., which might increase the effective area, or "theoretical outlet," to a like quantity and bear out the statement made by Mr. Snow. This "theoretical outlet," as has been stated by Mr. Snow, is not to be understood to be the actual sizes of the outlet of the fan's casing, but to be the size of opening which will offer a resistance equivalent to the sum of all resistances of distributing pipes. If in any given case we are able to state what this equivalent outlet is to be, we can then select the size of fan which will give us the desired volume of air at the desired pressure.

In Fig. 4 I have reproduced the efficiency curves and have drawn another curve showing the relation existing between the size of outlet and the ratio of air velocity to peripheral velocity of wheel. It is not safe to reason too much from the concrete to the abstract, but comparing this curve with similar ones constructed from the tests of a small pressure blower, as given in Mr. Kent's handbook, and from the guaranteed results of one blower-maker on his standard fans (presumably computed from tests which are not available to the public), I think it safe to say that the curve of the tests may be applied to different sizes of fans with reasonably satisfactory results.

To illustrate the advantages that have already been mentioned, we have only to notice, for instance, that the fan tested, when running at 200 r. p. m., will develop 1 oz. pressure when delivering a volume of 32,000 cu. ft. per minute, at an efficiency of 51% and requiring 17.2 I. HP. to run it. The same fan running at 161 r. p. m. will deliver the same volume of 32,000 cu. ft. at a pressure of ½ oz. and an efficiency of 41.7%, but requiring only 10½ HP. While the efficiency in the first instance is considerably higher, it should be pointed out that the actual horse-power required is 6.7 more. This represents the actual cost of delivering 32,000 cu. ft. of air into the room under the two pressures noted. Again, this fan running at 142 revolutions per minute will deliver 25,800 cu. ft. of air at a pressure of ½ oz., and at an efficiency of 54%. If, however, it is desired to deliver 32,000 cu. ft. of air at ½ oz. pressure and at maximum efficiency, it will be necessary to use a larger size of fan, the determination of which will illustrate the use which I have made of these tests.

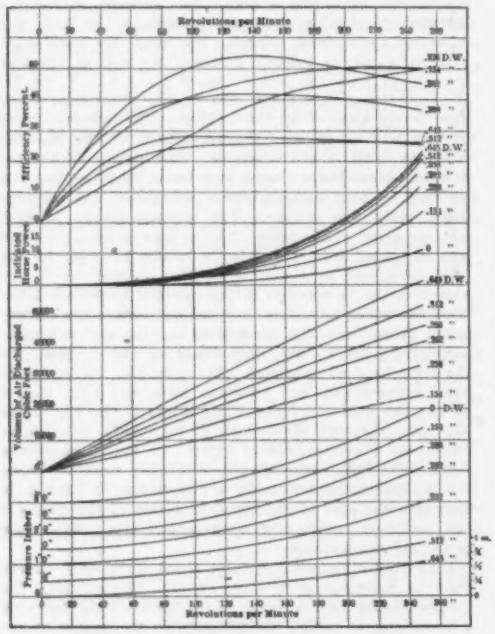


Fig. 2. Diagram Plotted from Results of Tests of 8-ft. Fan. (Abscissae Represent Speed of Fan in revs. per min.)

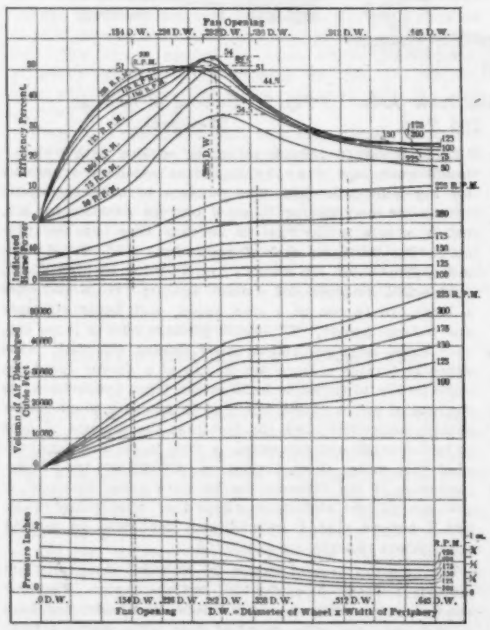


Fig. 3. Diagram Plotted From Results of Tests of 8-ft. Fan. (Ordinates Represent Speed of Fan.)

I am not prepared to say that for fans with inlet on one side only, 29 D. W. is as large a "theoretical outlet" as should be used. However, it seems to be evident that where inlet may be had on both sides, a wider fan may be used, and possibly further experiments may show that a wider fan may be used with a single inlet. Assuming that .29 D. W. is to be used, we have for the volume of air delivered,

$$Q = .29 \text{ D. W. } V,$$

in which V is the air velocity corresponding to the assumed pressure.

The velocity corresponding to the pressure of ½ oz. is 3653.8. From the foregoing equation we find that,

$$\text{D. W.} = \frac{32000}{3653.8 \times .29} = 30.2.$$

If $W = \frac{D}{r}$, then $D^2 = 30.2\ r$. The proper value of "r" was not determinable from these tests, but probably has been determined by the builders' experiments. It is apparently between 2.3 and 2.7. In the fan under test,

$$r = 2.595.$$

It is desirable to use standard diameters and vary the width within certain limits. In the present case the size of fan selected should be 8½ ft. diameter by 42.6 ins. wide at periphery. For the speed of fan, we have from the lower curve in Fig. 4 for .29 D. W.,

$$\frac{V}{V_p} = 1.025.$$

From this equation the peripheral velocity,

$$V_p = \frac{3653.8}{1.025} = 3570 \text{ ft. per min.}$$

This is equivalent to 133.5 r. p. m. of an 8½-ft. fan. By the efficiency of 54% shown by the chart, this would require 8.125 HP., a still further saving over the 8-ft. fan running 161 revolutions per minute.

There is another problem upon which I should be glad to have discussion and enlightenment. In designing an induced draft plant some time since, I proposed to install two fans, which, under normal conditions, would both run at slow speed, either of which in an emergency could be speeded up to do the work of both. One representative informed me that he did not think it could be done practically, and after I insisted that it be worked out, two 10-ft. fans, 24 ins. wide at the periphery, were offered. After some discussion and computations, two 10-ft. fans 54 ins. wide at periphery were purchased. I based my contentions on the tests which I have described, although I am aware that the conditions when handling gases at a high temperature are somewhat different. I do not see, however, that these changed conditions prevent the use of a fan in the way I described.

To illustrate, the following data may be taken from the curves of the 8-ft. fan tested. With an outlet of .265 D. W., and running at 150 r. p. m., the fan will deliver 25,000 cu. ft. of air at a pressure of 9-16 oz., requiring 7.8 HP. and give an efficiency of 53%. This same fan with an outlet .53 D. W., running at 225 r. p. m. will deliver 50,000 cu. ft. of air at the same pressure of 9-16 oz. In the latter case the horse-power is 40.8, and efficiency but 26.5%. This efficiency is very low, but is not to be considered in an emergency. The only question which requires particular attention is an arrangement which will allow a change of the outlet, or its equivalent resistance, from .265 D. W. to .53 D. W., and this was very easily accomplished in the case mentioned. It is necessary in many industrial plants to provide against a shut down, due to any ordinary accident, but I fail to see the necessity of putting in two fans, each sufficiently large to do

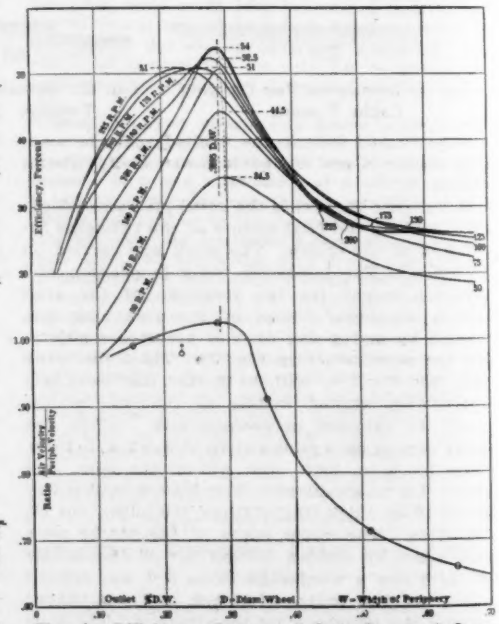


Fig. 4. Efficiency Curves of 8-ft. Fan and Curve Showing Relation Between Size of Outlet and Ratio of Air Velocity to Velocity of Tips of Fan Blades.

RESULTS OF TESTS OF 8-FT. FAN.
(Diameter of Wheel, 8 ft.; Width of Periphery, 37 ins.)

1	2	3	4		5		6		7		8	9	10	11	12	13	14
Size of Fan Outlet.	Speed, revolutions per min.	Peripheral velocity, feet per minute.	Vacuum at inlet.		Pressure in chamber.		Velocity head at orifice, by Pitot tube. h = inches.		Total head. C + D		Velocity of air, v = 60.74 √h ft. per min.	Velocity due to pressure in chamber, ft. per min.	B ÷ A	Cubic feet of air discharged per minute.	Theoretical horse-power.	Observed horse-power.	Efficiency per cent.
		A	C		D						B						
			Inches.	Ounces.	Inches.	Ounces.	Inches.	Ounces.	Inches.	Ounces.							
A .154 D. W. 2½ in. diam. 2.79 sq. ft.	44.2	1,109	.100	.057	.116	.066	.110	.063	.216	.123	1,294	1,202	1.15	4,868	.098	.330	29.6
	74.5	1,869	.111	.064	.217	.125	.197	.110	.326	.188	1,591	1,740	.90	6,411	.218	1.040	21.0
	109.0	2,510	.120	.069	.397	.229	.368	.209	.517	.298	2,329	2,254	.92	8,881	.551	2.000	27.5
	137.8	3,445	.154	.088	.903	.516	.826	.482	1.057	.604	3,343	3,526	1.05	13,420	1.880	4.27	44.0
	140.6	3,526	.156	.089	.858	.494	.822	.480	1.014	.583	3,530	3,490	1.02	13,990	1.865	4.92	38.7
	153.0	3,955	.227	.130	1.040	.600	.943	.543	1.393	.756	3,758	3,901	.97	14,940	2.345	5.91	39.5
	155.9	3,912	.220	.126	1.076	.615	1.047	.608	1.356	.780	3,960	3,848	1.01	15,010	2.515	5.98	42.1
	180.6	4,784	.281	.167	1.710	.984	1.630	.950	1.991	1.151	4,971	4,865	1.08	18,840	5.060	.....	.....
	192.6	4,824	.434	.250	1.803	1.036	1.518	.868	2.016	1.158	4,768	4,648	.98	18,090	4.430	10.34	42.8
	229.9	6,018	.477	.275	2.668	1.513	2.620	1.500	3.145	1.788	6,264	5,984	1.04	23,750	9.810	20.67	46.9
B .226 D. W. 3⅜ in. diam. 5.56 sq. ft.	49.4	1,289	.054	.031	.131	.075	.117	.067	.185	106	1,324	1,345	1.06	7,374	.151	.39	38.7
	66.8	1,676	.070	.040	.224	.129	.202	.116	.294	169	1,749	1,764	1.04	9,741	.343	.76	45.2
	108.8	2,730	.143	.081	.487	.280	.451	.260	.629	.351	2,598	2,596	.95	14,490	1.107	2.71	40.8
	109.5	2,774	.187	.107	.495	.285	.443	.255	.682	.392	2,586	2,600	.93	14,430	1.125	2.70	41.6
	130.5	3,624	.218	.126	.813	.353	.546	.316	.831	.478	2,960	2,913	.95	15,990	1.530	3.59	42.7
	134.5	3,375	.250	.144	.734	.424	.692	.398	.894	.568	3,319	3,195	.95	17,930	2.075	4.78	43.4
	158.2	3,970	.285	.164	1.210	.695	1.110	.640	1.495	.859	4,080	4,000	1.03	22,730	4.380	8.32	52.6
	161.2	4,046	.370	.213	1.057	.605	.957	.550	1.427	.818	3,786	3,817	.93	21,080	3.460	8.01	43.6
	173.4	4,347	.380	.219	1.441	.830	1.299	.748	1.721	1.049	4,411	4,478	1.02	24,670	5.860	.....	.....
	189.0	4,750	.700	.403	1.375	.795	1.319	.756	2.075	1.198	4,481	4,346	.93	24,680	5.905	11.84	44.6
	202.8	5,090	.410	.236	2.000	1.145	1.811	1.045	2.410	1.381	5,308	5,245	1.02	29,420	9.060	16.21	55.8
C .282 D. W. 2½ in. diam. 6.90 sq. ft.	51.1	1,275	.070	.040	.140	.085	.142	.081	.210	.125	1,458	1,432	1.14	10,180	.236	.71	33.3
	76.0	1,907	.100	.057	.281	.163	.292	.168	.391	.229	2,055	1,983	1.07	14,220	.637	1.55	47.25
	108.8	2,580	.192	.112	.476	.274	.478	.275	.668	.386	2,676	2,580	1.05	18,650	1.395	2.81	49.6
	115.0	2,898	.200	.149	.605	.347	.607	.350	.855	.496	3,015	2,952	1.04	20,980	1.990	3.82	52.1
	136.8	3,430	.293	.169	.816	.470	.813	.468	1.109	.639	3,496	3,395	1.02	24,330	3.123	5.53	56.5
	151.3	3,797	.290	.163	1.097	.577	1.004	.577	1.387	.749	3,577	3,736	1.04	27,050	4.380	7.06	53.3
	177.4	4,452	.480	.277	1.498	.825	1.498	.830	1.978	1.102	4,634	4,455	1.04	32,180	7.945	13.53	52.8
	187.8	4,713	.550	.317	1.479	.845	1.457	.835	2.029	1.162	4,671	4,509	.99	33,500	7.490	15.55	48.2
	196.4	4,929	.700	.404	1.699	.933	1.694	.950	2.319	1.337	4,939	4,736	1.00	34,360	8.750	17.65	49.5
	216.0	5,421	.716	.413	1.991	1.140	1.947	1.120	2.707	1.558	5,460	5,284	.99	37,980	11.79	22.97	51.7
D .338 D. W. 3½ in. diam. 8.85 sq. ft.	47.8	1,199	.040	.023	.102	.058	.102	.058	.141	.081	1,046	1,155	1.04	11,080	.174	.43	38.8
	73.5	1,819	.070	.040	.194	.111	.192	.110	.264	.151	1,695	1,636	.93	15,010	.455	1.30	35.0
	99.0	2,459	.080	.046	.340	.196	.341	.196	.420	.242	2,260	2,174	.92	20,010	1.070	2.80	38.2
	114.8	2,856	.410	.236	.479	.275	.417	.275	.689	.511	2,673	2,581	.94	23,670	1.775	4.24	41.8
	117.3	2,941	.390	.167	.498	.284	.457	.263	.748	.431	2,616	2,583	.89	23,360	1.675	4.35	39.4
	131.1	3,229	.400	.231	.613	.352	.606	.350	1.013	.583	3,004	2,913	.93	26,660	2.555	5.19	49.2
	140.4	3,504	.500	.288	.745	.428	.740	.426	1.246	.716	3,329	3,212	.94	29,470	3.445	8.19	42.1
	151.1	3,732	.485	.279	.746	.428	.749	.430	1.231	.707	3,349	3,212	.90	29,650	3.460	8.91	38.8
	167.6	4,206	.500	.288	.875	.505	.880	.507	1.375	.793	3,630	3,488	.86	32,140	4.430	10.89	40.9
	189.6	4,734	.....	.....	1.160	.670	1.147	.660	.....	.....	4,144	4,016	.87	36,690	6.700	.....	.....
	191.4	4,804	.710	.409	1.270	.730	1.258	.720	1.969	1.129	4,532	4,165	.90	38,300	7.035	16.08	41.7
E .512 D. W. 4½ in. diam. 12.65 sq. ft.	69.4	1,731	.060	.051	.136	.078	.133	.076	.226	.129	1,411	1,372	.81	17,850	.3795	1.31	28.9
	106.9	2,682	.230	.132	.366	.152	.260	.149	.496	.284	1,973	1,915	.73	24,960	1.035	3.76	27.5
	185.6	3,403	.374	.215	.413	.238	.403	.233	.787	.453	2,466	2,396	.72	31,200	2.085	7.14	29.4
	152.7	3,844	.456	.263	.599	.291	.496	.286	.945	.544	2,709	2,663	.71	34,380	2.630	9.34	28.2
	175.6	4,407	.630	.364	.644	.373	.638	.367	1.274	.736	3,102	2,995	.70	39,380	3.960	14.80	26.9
	210.0	5,271	.860	.495	.902	.520	.893	.513	1.702	1.015	3,657	3,589	.69	46,280	6.560	24.93	26.3
F .645 D. W. 5¼ in. diam. 15.90 sq. ft.	80.7	2,025	.120	.069	.136	.078	.133	.076	.256	.147	1.411	1,372	.70	22,430	.478	1.82	26.2
	117.1	2,938	.365	.211	.252	.144	.211	.138	.617	.355	1.900	1.874	.65	30,180	1.185	5.20	22.7
	128.7	3,079	.442	.255	.293	.168	.261	.163	.735	.423	2.051	2.019	.67	32,560	1.495	6.43	23.5
	164.1	4,118	.790	.456	.479	.287	.497	.273	1.269	.743	2,673	2,631	.63	42,490	3.330	14.10	23.6
	194.7	4,886	.944	.543	.634	.367	.597	.343	1.582	.910	2,990	2,975	.61	47,580	4.750	19.36	24.5
	202.4	5,080	1.130	.652	.772	.477	.719	.414	1.902	1.129	3,441	3,300	.68	54,680	7.110	24.48	29.0
G 6-D. W. outlet closed	51.3	1,299	.....	.....	.200	.115	.....	.....	.....	.....	.....	1,666	1.28	.....	.....	.34	.....
	72.4	1,819	.....	.....	.297	.171	.....	.....	.....	.....	.....	2,032	1.11	.....	.....	.59	.....
	96.8	2,463	.....	.....	.477	.276	.....	.....	.....	.....	.....	2,580	1.04	.....	.....	1.08	.....
	129.2	3,247	.....	.....	.557	.318	.....	.....	.....	.....	.....	2,770	.85	.....	.....	1.95	.....
	140.0	3,518	.....	.....	.936	.540	.....	.....	.....	.....	.....	3,608	1.02	.....	.....	2.21	.....
	151.7	3,812	.....	.....	1.007	.633	.....	.....	.....	.....	.....	3,906	1.02	.....	.....	2.77	.....
	176.0	4,423	.....	.....	1.571	.907	.....	.....	.....	.....	.....	4,673	1.06	.....	.....	4.33	.....
	208.3	5,109	.....	.....	2.051	1.184	.....	.....	.....	.....	.....	5,535	1.04	.....	.....	6.08	.....
	226.8	5,699	.....	.....	2 481	1.453	.....	.....	.....	.....	.....	5,867	1.03	.....	.....	8.11	.....

the entire work under normal conditions, at maximum efficiency.

I have not entered into this discussion with any feeling akin to the old Quaker's who is reported to have told his wife that "all the world is queer except thee and me, and thee is a little queer." But I have hoped to provoke a discussion by those who have had better opportunities for studying the problem than I have, and can give us information not obtainable, at least convincingly, from these tests.

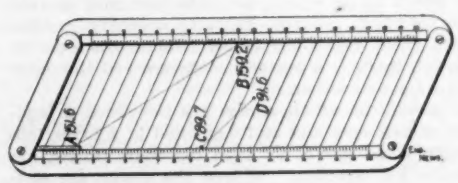
In closing I wish to acknowledge the able assistance and painstaking care of Mr. C. W. Wilder, who made most of the observations and computations for me. But for my confidence in his ability, I should have hesitated to present these results to the Society.

AN INSTRUMENT FOR INTERPOLATING CONTOURS.

About three years ago there was a lively correspondence in the columns of Engineering News on methods and appliances for interpolating contours between observed points on a topographical map (see Eng. News, March 29, May 10, June 21, July 26, and Aug. 16, 1900). Some of our correspondents described instruments or diagrams in various modifications, others recommended the use of a slide rule for computing the contour intervals. We note in the following a device for this purpose which we think is an improvement on those described. Its main advantage over a diagram is that no reading of distance is required, so that its use is as nearly mechanical as possible; this, of course, would tend to minimize the chance of error. On the other hand, it has the advantage over the non-reading instruments described by our correspondents, that the contour points are marked through the instrument directly on the map, no transferring of distances being required.

The instrument is quite simple in construction; essentially it is nothing more than a scale of proportional parts which may be expanded or contracted to suit different distances. It consists of two parallel scales held apart by two links of equal length, just like an old form of parallel ruler. The corresponding division points of the two scales are joined by threads, which give a parallel ruling of uniform interval. The angle of the whole parallelogram can be changed by swinging one scale relative to the other, the links acting as parallel-motion distance-pieces. The threads are always held taut, as the pivots are in line with the edges of the scales.

The use of the instrument is made quite clear



Instrument for Interpolating Contours.

by the following example. Suppose A and B are two adjacent observed stations on the map, their elevations being 151.6 ft. and 159.2 ft., and 1-ft. contours being desired. Then place the point 1.6 of the lower scale on A, and using this as a center, swing the instrument around, at the same time swinging the upper scale on the links, until scale-point 9.2 on the upper scale coincides with the plotted position of station B. Then a line drawn from A to B will be intersected by the threads at the contour-points 152, 153, 154 . . . 159, which points may be marked on the drawing. The use of the instrument in other cases will be obvious. A special case is shown in the sketch by the points C and D, which differ but little in elevation and are too near together to permit the use of the two scales; in this case C is taken on the lower scale and D on an imaginary scale a little above the lower one, using the threads as guides in estimating the scale-divisions. It will be possible in most cases to take for the lower scale-point some point in the first or second division on the lower scale, and to facilitate the swinging of the instrument around the selected point a small plate is provided at the first two scale divisions with pinholes at the separate tenths; in the example given above, the point 1.6 on the lower scale is set on A, and a pin is stuck in the paper at this point, which serves as a center for swinging the instrument.

This instrument was devised by Mr. J. Kubala, an Austrian engineer, and is described in the "Zeitschrift des Oesterreichischen Ingenieur und Architekten Vereins." It is made by the firm of Neuhöfer & Son, of Vienna; its size is 17 × 7 cm., about 6¾ × 2¾ ins.

THE WATER SUPPLY OF MANILA, P. I.*

Request has been made by the natives living in the valley of the Mariquina river, from which the water supply of Manila is obtained, that the present sanitary cordon around the river be withdrawn and that they be given access to it. At present the watershed is carefully guarded by American soldiers, and pollution of the river prevented as far as possible. The natives argue that as cholera no longer exists in the valley there is no reason why they should not be allowed to use the stream as they had previously been in the habit of doing. The careful quarantine of the Mariquina valley is undoubtedly the reason why Manila has practically been exempt from cholera for the past four months, and with cholera existing at a large number of points in the islands there is no possibility that the request for the removal of this quarantine will be entertained for a moment by the health authorities. The quarantine of the river was removed for a short time last November, but this action was followed by an alarming increase in the number of cholera cases in Manila, and the quarantine was promptly re-established, with an immediate reduction in the cholera mortality of the city.

The water supply of Manila offers one of the most urgent sanitary problems at the present time. In quantity it is insufficient for the existing population of the city, allowing only about 10,000,000 gallons daily, or a total of about 30 gallons per capita for all purposes. The mains are so small and the reservoir at such slight elevation that the pressure is insufficient to force water to the second stories of buildings in many parts of the town during business hours, even when the pumps are run to their full capacity. With the rapid enlargement of the city which is going on under the stimulus of increased business due to American control of affairs, and particularly when a proper sewerage system for the city is installed, the amount of water necessary for city uses will have to be doubled. Such an increased amount will mean practically the entire flow of the Mariquina River at the dry season. At present, a flimsy dirt dam, liable to be washed out at any time by a sudden flood, forms a small pond from which the water is pumped by four engines, of obsolete type, to the top of a hill, about 140 ft. high, whence it runs by gravity into a reservoir at El Deposito, where a certain amount of purification by sedimentation is accomplished. In the dry season these four engines are just capable of satisfying the city's needs, and if one or two of them should be crippled for any length of time a water famine in Manila would result. The reservoir at El Deposito holds a supply for about thirty-six hours only, an amount quite insufficient to tide over even an ordinary emergency. The Deposito reservoir is divided into two parts to permit of alternate cleaning; it consists of a large number of connected vaults hewn out of the solid rock and arched over with masonry, on a hill about 65 ft. higher than the level of Manila. The water is carried into the city from the reservoir by gravity, being taken over the intervening four miles through a large pipe laid on the surface of the ground. In character, the water supply of Manila is not at all satisfactory, as at present some 20,000 natives live on the Mariquina watershed above the intake for the pumping station, and the town of Mariquina, directly on the river banks, is only about a mile and a half from this pumping station. Under such conditions absolute freedom from contamination cannot be secured even by the most strict quarantine measures, especially during the rains. The government is fully alive to the necessity of providing a more abundant and purer water supply, and a corps of engineers is engaged in drawing plans and making up estimates for this purpose.

*From a letter from the Manila correspondent of the New York "Medical Record," dated March 15, 1903, and published in the "Record" for June 13, 1903.

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The prevalence of mosquitoes in the suburbs of New York during the past week or two has been rather discouraging to some, at least, of the local boards of health that have been fighting the pest. It should be remembered, however, that the previous frequent rain and the warm weather have been particularly favorable to mosquito breeding, creating and maintaining shallow pools by the thousand in localities where such rarely exist.

The unpleasant conditions that have prevailed illustrate anew the desirability of depending on surface drainage rather than crude oil for the suppression of mosquitoes, and of looking out for small pocket holes, for accumulations of old tin cans, and for manure pits at suburban stables. Finally, there is much consolation in the undoubted fact that in most localities the malaria-spreading mosquitoes form a very small proportion of those abroad at any time, and that there is reason to believe that intelligent work directed against their breeding places is particularly effective.

The report on the proposed Charles River dam, to which several references have been made in past issues of this journal, has been published. A notice of it will be found in the Engineering Literature Supplement which accompanies this issue. There is also reprinted in the same Supplement the recent paper before the Society for the Promotion of Engineering Education, in which Mr. J. A. L. Waddell outlined a plan for a History of Engineering. In the Construction News Supplement there is a further discussion of Cost of Overhaul, including a letter from a contractor; also some comments on attempts to throw upon contractors all the risks of material of unknown character.

The paper describing tests of a fan blower, printed elsewhere in this issue, offers the same practical conclusion that has been several times expressed editorially in these columns, viz.: In moving air, keep velocities and pressures low in order to work with least expenditure of power. The volume of air to be moved is usually a fixed quantity and it can be either moved with high velocity and much expenditure of power or with low velocity and small expenditure of power.

This seems almost self-evident to anyone with even an elementary knowledge of mechanics, yet nothing is more common than to see installations in which power is wasted by the use of small high-speed fans and air conduits. Because a small machine, and small pipe can be made to do the work and will cost less to install, the daily loss of power is frequently overlooked.

Besides this, the high-speed machine is objectionable from other points of view. It is much more likely to give trouble from hot bearings; it wears out faster, is much more noisy and the drafts produced at the point of delivery are often a subject of complaint. Practice is tending all the time, therefore, toward the use of larger and slower speed machines for blower installations.

Chicago appears to be having another one of its periodic failures to secure funds enough to clean its streets and remove its garbage. At least the local papers quote Public Works Commissioner Blocki as saying, the latter part of June:

> There will be no streets cleaned after July 30. In fact, street cleaning is stopped now, and I doubt if there will be any more of it done this year—except in the First ward.

A similar story comes from Chicago so often as to raise serious questions regarding the generosity of the taxpayers in approving expenditures for municipal cleanliness, or else the financial or administrative ability of the city government. True, other cities fall short of any very high ideal in street cleaning and garbage disposal matters, but such stories as this do not come again and again from any other large city. It is at least charitable to say that this is one of the penalties of bigness, and that with so rapid a growth in population, area, and public needs, it is next to impossible for Chicago to meet all the wants of its citizens. This suggests a few serious questions: Is not the city in sore need of reforms in local taxation, in financial administration, and in both street cleaning and garbage disposal methods? These questions we must leave for the press and people of that city, and to them we will also submit this further question: Is it not time for most Chicagoans to change their ideals for their city to a less grossly material basis, substituting for mere bigness some of the things that make for true civic greatness? Among these might be better and cleaner pavements, less smoke and a proper sanitary supervision of the milk supply.

THE MERITS OF RUBBLE CONCRETE.

There is a type of masonry that has developed within the last few years, and about which very little has yet been printed. We refer to concrete in which large stones are imbedded and to rubble in which concrete replaces ordinary mortar. Both these types are called rubble concrete, although there is a material difference in the manner of construction.

Concrete in which stones are imbedded is usually mixed very wet and deposited in a layer into which large spalls, or even one-man stones, are driven by ramming. Where the spalls are flat-bedded, a dry-mixed concrete is sometimes used, the spalls being laid on their flat face and concrete rammed into the vertical joints. It is but a step from the last-mentioned method of wall building to the rubble concrete in which large irregular stones are deposited in the wall by means of a derrick, the joints between these blocks being afterward filled with concrete—preferably wet-mixed. This last form of masonry was illustrated in our issue of June 18, in an article on the Spier Falls dam.

For small retaining walls and for comparatively thin foundations the method of ramming large spalls into wet-mixed concrete is doubtless to be chosen. In massive masonry, where the quarries yield large blocks of stone, it would be absurd to break all the stone up into spalls and concrete sizes. In such cases the blocks should be bedded in soft concrete and the vertical joints filled with soft concrete into which spalls may be rammed.

We are aware that many engineers will cling to the use of masonry laid in dry-mixed mortar or to plain concrete, and we concede that there are legitimate uses for both.

Concrete possesses three distinct advantages over ordinary stone masonry: (1) It does not require stone-cutters to shape it; (2) it does not require skilled masons to lay it; (3) it may be made of shaley stone and of gravel. Concrete, however, possess these decided economic disadvantages: (1) It requires the installation of a stone-crushing plant; (2) it usually requires frames or molds; (3) it requires more cement per cubic yard than any other class of masonry.

Let us consider the relative weights of these advantages and disadvantages. To begin with, the fact that concrete dispenses with stone-cutters generally outweighs all other considerations. But let us keep clearly in mind that the more massive the masonry the smaller is the percentage of face or cut stone. In retaining walls, in piers and in arches the proportion of cut stone is very high, the arch, for example, being all cut stone. In dams, however, the cut stone facing generally forms a very small part of the masonry yardage, so that the cost of the backing will practically determine the cost of the dam.

Second-class retaining walls on the Erie Canal required about 0.6-barrel of Portland cement per cubic yard of masonry, the mortar being 1 to 2; whereas a 1-2-5 concrete (packed measure) consumed 1.1 barrels of cement per cubic yard of concrete. With Portland cement at $1.60 a barrel, we have a difference of 80 cts. a cu. yd. in favor of the rubble. In addition, there was a saving of about 50 cts. a cu. yd. in the item of forms where rubble was used. Besides this there was the saving in crushing the stone, or, say, 30 cts. a cu. yd., thus making a total of $1.60 per cu. yd. in favor of the rubble. The cost of laying the rubble was about 80 cts. per cu. yd., using high-priced masons, as against 60 cts. per cu. yd. for mixing and laying concrete by hand, or 40 cts. by mechanical mixers. These costs of laying indicate quite clearly that backing masonry costs very little more to lay than does concrete. As a matter of fact, where rubble concrete is used no skilled masons need be employed at all, so that this slight advantage of concrete over rubble is lost entirely. The drift of our argument is now apparent, namely, that, except in places where cut stone would be used, concrete has ordinarily no economic advantage over stone masonry, and is at a positive disadvantage as a backing where appearance is a factor of no moment. We are not now speaking of localities where gravel exists and stone does not.

On the New York Central R. R. many of the old masonry retaining walls have been recently faced with a foot or more of concrete. This has been done to save the cost of frequent repointing of joints and to protect poor stone from disintegration. The question now arises whether such walls should not be built all at once and not piecemeal. In a word, should we not give up the wholesale and indiscriminate use of concrete, now so common, and use the concrete only where we formerly used cut stone facing, and as a mortar in the rubble backing? Reinforced concrete may be regarded as a structural material by itself, and we are not now speaking of the comparatively light structures in which it is used, but rather of those structures that depend upon their inertia for their stability.

Where stone comes from the quarry with natural flat beds, common mortar may well continue to be used in the bed joints, reserving the concrete for filling the vertical joints so as to avoid dressing the stone at all. Where stone comes out in slabs easily broken by a hammer, or where retaining walls are too thin to permit the use of large blocks, a soft concrete into which the small stones are rammed may be used. Where stones come out in large, tough, irregular blocks,

then undoubtedly a true rubble concrete, such as was used at the Spier Falls dam, is the cheapest form of backing masonry; for even using skilled masons the cost of laying is but slightly greater than mixing and laying concrete with common labor.

In properly-designed retaining walls and dams the tensile strength of the masonry is a factor of minor importance, for even a dry wall, properly bonded, would resist sliding or overturning. The mortar serves primarily to secure the bond that prevents the inner stones from producing a lateral thrust on the outer ones. If this view of the function of the mortar is correct, then a rubble wall that will uphold its own weight when subject to jars or shock, will resist overturning when designed so that the resultant thrust falls within the middle third.

Should the designer, for any reason, insist upon great strength, rubble concrete still stands upon a better footing than concrete. For if the cement saved by imbedding large stones in the concrete be used to make a richer mortar, the rubble masonry will unquestionably be superior in strength to pure concrete having a weaker mortar, the weakest link in the chain ordinarily being the mortar. It is evident that a dollar will buy either more rubble concrete masonry than concrete, or it will buy a stronger masonry; in the one case adding to the factor of safety by increasing the inertia,, in the other case by increasing the strength.

LETTERS TO THE EDITOR.

Burning Illinois Coal Without Smoke.

Sir: Referring to the extract in your issue of June 11, of the paper read by Professor Breckenridge before the Commercial Club of Chicago on the subject of burning bituminous coal without smoke, it is gratifying to observe that the professor of mechanical engineering of an important university is disposed to give the matter some thought.

This communication is not intended as a criticism of the paper, but it is my intention to present some views along the same lines, therefore I will follow the arguments. It is stated that the volatile matter must be distilled from the coal slowly, but this distillation must necessarily proceed at the same rate as the feed of the coal. Therefore it is desirable that the feed of the fuel, and consquently the distillation of the hydrocarbon gas, proceed at a uniform rate; this is necessary because the flow of the air supply is uniform. This result is secured by the automatic stoker, and its degree of success is due to it. If coal of a corresponding size of that used by a stoker is fed at a uniform rate evenly over the fire of an ordinary flat grate, the result will be as good as that of the stoker, the only limitation being that it may be necessary to interrupt the process to clean the fire, while the stoker may be of a type that is self-cleaning. This uniformity of the feed of the fuel is the first step in securing the thorough mixture of the combustible and air that is absolutely necessary if a smokeless condition is obtained.

The second requirement given is that the distilled gases must be intimately mixed with hot air. This reference to "hot air," I think, is liable to convey the impression that pre-heated air is more effective, and to suggest the desirability of special heaters for raising the temperature of the air supply before its entrance to the furnace. The whole secret of a smokeless condition lies in the intimate mixture of the gases and air, and it is probable that the necessary elevation of temperature required for the union of carbon with oxygen is often overestimated. A familiar example will show that it occurs at temperatures which are not "high" as the term is generally used; in the case of a blast furnace the gases leave the top at a moderate temperature, and after passing through long lines of pipe the carbon monoxide readily ignites in the boiler furnaces. It is possible that high temperature may have some effect in promoting motion among the particles of the gas volume, and in this way cause better mixture, but if this is true the effect cannot be great. On the other hand, it is probable that the effect of high temperature on a mass of hydrocarbon gas is to break up the combination to a greater extent than would a lower temperature, resulting in a greater amount of carbon going free, provided oxygen is not available for combination with it.

The second and final step in finishing the combustion consists in obtaining perfect mixture of the gases in the furnace, and this mixture must be secured while the temperature is sufficiently high for the oxidation of carbon. If a smokeless condition is produced this second step will amount only to putting the finishing touches on the work partially accomplished at the fire. If each molecule of gas on its liberation at proper temperature could immediately meet its required supply of oxygen the problem would be immediately solved; this, however, is impossible. The tendency is for the gas to flow from a piece of coal in a compact mass, and nitrogen and carbon dioxide must flow away that nitrogen and oxygen may take its place to continue the required oxygen supply. If the piece of coal is large, more time is required, or if the coal is very small in size and the bed of considerable thickness it is then in effect quite similar to a large solid mass.

The only kind of furnace that is "smoke proof" is the combination of a perfect feeding stoker with a sufficiently large mixing chamber. Such kind of stoker is illustrated by the chain or traveling grate; with it the operator cannot cause the feed of coal to be otherwise than as intended.

No fixed set of rules can be offered for the smokeless operation of various furnaces, using different fuels. Each case generally requires individual study unless it be of the smoke-proof kind. For example, it is stated in the paper that a strong draft is desirable, but in the two following examples a strong draft produces opposite results. With a chain-grate stoker under a boiler so arranged that the gases flow immediately among the tubes of the boiler, a strong draft is very effective in reducing smoke. With a Hawley furnace of 132 sq. ft. of both upper and lower grates, where the fire was carefully sliced at brief intervals, and leveled on the bottom grate at each slicing, there was almost no smoke. But owing to the strength of draft the capacity ran as high as 850 HP.; an impossible amount of work for one man except for a short interval only. The result was that the men poked at occasional intervals and accumulated a considerable thickness of fire on the bottom grate; much smoke was produced, but the capacity was within reason. It was only after the draft had been reduced that it was possible to maintain a condition without objectionable smoke.

The general tendency in matters relating to the operation of boilers and furnaces is to assume causes as an explanation of the various phenomena, rather than to arrive at conclusions by means of detailed study and investigations. The result is that much mere speculation passes current as fact. And it is probable that the furnace and boiler salesman is oftener responsible for the features of design, than is the engineer who makes the drawings. In mentioning the need of intelligent direction in boiler plants, Professor Breckenridge has touched on the most important point of the whole matter. It is very important, but the talent is not available, because the best quality of boiler-room labor is not sufficiently educated, or of the right character, to furnish any but occasional specimens competent to direct the operation. And the educated engineer as a rule either considers the details of the boiler plant as beneath his dignity, or is unwilling to take the trouble to acquire for himself a sufficient understanding of the problems involved, or to train the men under him. It is necessary that the coal-burning business be made more fashionable, and such a movement should start in the engineering colleges. In my work, in a number of cases, I have had young engineers with three or four years' experience in general work after leaving college, placed in large and important boiler plants, first as special students, then in charge of operation and maintenance. In each case the compensation was liberal, and as far as technical training was concerned the men were competent. Experimental apparatus and laboratory was available. In every case the man was wanted for what would be considered a better position at larger salary, as soon as he had accomplished the work and organization of the plant and broken in his successor. But none of them could be brought to a realization of the importance of the work, and the efforts resulted in failure, as far as benefit to the employers was concerned.

Most furnaces can be operated so that they will make but little smoke; sometimes, however, the great increase in labor cost, or reduced efficiency, justifies changes in the apparatus; the majority, however, only require proper manipulation to secure improvement. I have found a detailed study of each case valuable, and to this end have made profitable use of frequent determination of carbon dioxide, which, plotted as a curve in connection with notes designating the various manipulations of the furnace, or fire, affords a graphic representation of the process which is of much value. Most of the college boiler plants are equipped with an econometer; the result of my observation is that it is seldom used, and it appears usually to be regarded as a scientific toy. I would suggest that students in connection with their gas analysis work, take readings from this instrument at one or two minute intervals for a period of one or two hours, with different manipulation of fire, and then attempt an explanation of the irregularities of the curve. This, I think, will be more useful in giving an idea of furnace performance than the laboratory work with the Orsat and Hempel apparatus. Yours truly, A. Bement.

Chicago, June 25, 1903.

The Effect of Water on Asphalt Surface Mixtures.

Sir: Will you allow me to correct a statement in your issue of July 9, attributed to me in your account of the proceedings of the cement and bitumen section at the with the discussion of Mr. Dow's paper on bitumen, you say: "All asphalts, said he (Richardson), when exposed to the continual action of water will rot." This should read: All asphalt surface mixtures. Many asphalts in their pure form are not acted upon by water.

I also mentioned that at the present time the grading of asphalt surface mixtures was so much more carefully done that water had not the same chance to penetrate and act upon them as in the early days of the industry.

I would also suggest that you did not bring out the important fact that the experts from the Road Material Laboratory, of the U. S. Department of Agriculture, did not agree with the statements of Mr. Dow as to the cause of the action of water on asphalt surface mixtures. Mr. Allerton Cushman, chemist of that laboratory, stated that he could not believe that this could be attributed to the presence of as little as 0.3% of soluble matter, which is all that is found in Trinidad lake asphalt, while Mr. Logan Waller Page, Chief of that laboratory, suggest that illuminating gas from leaky mains had quite as much to do with the deterioration of pavements as water alone.

As Mr. Dow's views are being widely spread throughout the country it seems but fair that the views of other experts on this subject should be given equal publicity, although it is probable that the subject is one of but passing interest. Very truly yours,

Clifford Richardson,
Director New York Testing Laboratory.

Long Island City, N. Y., July 10, 1903.

The Recent Heavy Rainfall in New York City.

Sir: The following item may be of interest in the line of supplying additional data as to high rates of rainfall. The record is taken from an automatic rain gage, located on the top of a high building at New Brighton, in the Borough of Richmond, New York City.

On Monday, June 29, at 4.36 a. m., the rain commenced, ending at 1.30 p. m.; total time of rainfall was 8 hours and 54 minutes, and the total precipitation was 2.99 ins. The maximum downfall for one hour occurred between 9 and 10 o'clock, during which time 1.46 ins. were recorded by the gage. During that hour, between 9.33 and 9.51, the highest rate of fall was 4 ins. per hour, sustained for three minutes; the second highest rate, 3 ins. per hour for ten minutes; the third highest rate, 2.4 ins. per hour for five minutes, or a total for the lapsed time of 18 minutes of 0.9 in., or an average rate of 3 ins. per hour.

Inasmuch as this storm occurred at the close of a month when 6.98 ins. had already been recorded, and the ground was so saturated with water and the evaporation had been so slight during the period, our conclusion is, therefore, that the runoff of this particular storm of June 29 was very nearly equal to the amount of rainfall; consequently but few culverts and catch-basins were able to handle the fall without choking.

Yours very truly, Louis L. Tribus.

Commissioner of Public Works, Borough of Richmond.

New Brighton, N. Y., July 10, 1903.

Steel Dams and Concrete-Steel Diaphragms for Earth or Rock-Fill Dams.

Sir: I was sufficiently interested in the correspondence relative to a steel dam for adoption by the Reclamation service of the United States Geological Survey, and a rock-fill dam with metal-concrete facing suggested by your Kernville, Cal., correspondent (Eng. News, June 11 and July 2, 1903), to feel that an additional word on the subject may be useful and perhaps draw forth other and more valuable ideas on similar lines.

I have every reason to doubt that the engineers in charge of so great and varied a class of storage works as must be constructed under the Reclamation act would adopt any hard and fast type of dam, much less one so untried and so evidently ephemeral as a steel structure. I know that they contemplate at least one great masonry dam in Arizona, and also a huge earth dam in Montana, and believe that those in charge thoroughly realize the necessity for the most conservative designs and most permanent construction in government works.

I have given some thought of late to the availability of a thin diaphragm of the best metal-concrete construction as a suitable barrier to the passage of water through an earth or rock-fill dam. I believe that such a barrier will give results in many respects superior to an earth puddle or masonry core-wall in an embankment, or to a wooden facing or metal plate in a rock-fill.

I am convinced that such diaphragm should not be placed as suggested on the slope of the dam is an impervious facing. It should be built up vertically in the center of the dam as a wall of 4 to 6 ins. thickness throughout and firmly anchored in cement masonry at the foundation and up the abutments. Such a core diaphragm would be impervious, tough and flexible, if not too thick. It would be ideal in an earth embankment, and might prove equally successful in a rock-fill. I hope that some one may try it.

Yours truly, H. M. Wilson,
U. S. Geological Survey.

Washington, D. C., July 8, 1903.

The Design of Gusset-Plates.

Sir: I am unable to find articles in standard texts or journals which give full directions and explanations how to design gusset-plates in riveted steel roof-trusses or in similar construction. I wish up-to-date information covering the proportioning, the theoretical and practical rules, and a few illustrations. Specifications relative thereto would be greatly appreciated. Will you give me references or else submit my inquiry to the practice?

Very truly yours, C. F.

New York, July 11, 1903.

(We are not aware that the design of gusset-plates has ever been discussed in detail. Where only direct stresses, tension or compression, have to be transmitted there would seem to be little difficulty in the matter. But often a bending moment is introduced into the connection, and in calculating or estimating its effect on the stresses in the gusset-plate, the designer finds much room for the exercise of engineering judgmnt. The lateral stiffness of gusset-plates may be mentioned here as a point which is always neglected, and which in some cases at least should receive consideration. Any information relative to this matter of designing gusset-plates will, no doubt, be appreciated by others besides our correspondent.—Ed.)

Notes and Queries.

In our report of the recent summer meeting of the American Society of Mechanical Engineers at Saratoga, N. Y., we stated that there was no discussion of the paper by Mr. W. A. Stichter on "Alternating-Current Motors for Variable Speed." Mr. G. W. Colles, of Montreal, Canada, informs us that this report was in error, as he submitted a discussion (by letter). Mr. Colles considers the place of the alternating-current motor in the general field of motor-driving, and then reviews the different methods of variable-speed operation of alternating-current motors. The general conclusions are expressed in the last paragraph, as follows:

This question of finding a variable-speed motor to run on polyphase circuits is no academic question, but one which the mechanical engineer will have to face, and the electrical engineer to solve, in the near future. At the present rate of development it is clear that within a few years more our railways and elevators will all be run on polyphase or alternating circuits, and some substitute for the direct-current motor must necessarily be found.

A STANDARD METHOD FOR THE ANALYSIS OF CEMENT.

In the report of the Committee of the American Society of Civil Engineers on "Uniform Tests of Cement" (published in our issue of Jan. 29), it was recommended the chemical analyses of cement should be made according to the method proposed by the Committee on Uniformity in Analysis of Materials for the Portland Cement Industry, which was published in the "Journal of the Society of Chemical Industry" for Jan. 15, 1902.

As that journal is not generally available to engineers, we have thought it worth while to reprint this method. Through the courtesy of Mr. Clifford Richardson, of the New York Testing Laboratory, we have been furnished with a copy of this method, with such corrections as have been made in it up to the present time, and we present it as follows:

METHOD SUGGESTED FOR THE ANALYSIS OF LIMESTONES, RAW MIXTURES AND PORTLAND CEMENTS BY THE COMMITTEE ON UNIFORMITY IN TECHNICAL ANALYSIS WITH THE ADVICE OF W. F. HILLEBRAND.

SOLUTION: One-half gram of the finely powdered substance is to be weighed out and, if a limestone or unburned mixture, strongly ignited in a covered platinum crucible over a strong blast for 15 minutes, or longer if the blast is not powerful enough to effect complete conversion to a cement in this time. It is then transferred to an evaporating dish, preferably of platinum for the sake of celerity in evaporation, moistened with enough water to prevent lumping, and 5 to 10 c. c. of strong HCl added and digested, with the aid of gentle heat and agitation, until solution is completed. Solution may be aided by light pressure with the flattened end of a glass rod.* The solution is then evaporated to dryness, as far as this may be possible on the bath.

SILICA: The residue, without further heating, is treated at first with 5 to 10 c. c. of strong HCl which is then diluted to half strength or less, or upon the residue may be poured at once a larger volume of acid of half strength. The dish is then covered and digestion allowed to go on for 10 minutes on the bath, after which the solution is filtered and the separated silica washed thoroughly with water. The filtrate is again evaporated to dryness, the residue, without further heating, taken up with acid and water and the small amount of silica it contains separated on another filter paper. The papers containing the residue are transferred wet to a weighed platinum crucible, dried, ignited, first over a Bunsen burner until the carbon of the filter is completely consumed, and finally over the blast for 15 minutes and checked by a further blasting for 10 minutes or to constant weight. The silica, if great accuracy is desired, is treated in the crucible with about 10 c. c. of HFl and four drops of H_2SO_4 and evaporated over a low flame to complete dryness. The small residue is finally blasted, for a minute or two, cooled and weighed. The difference between this weight and the weight previously obtained gives the amount of silica.*

Al_2O_3 and Fe_2O_3: The filtrate, about 250 c. c., from the second evaporation for SiO_2, is made alkaline with NH_4OH after adding HCl, if need be, to insure a total of 10 to 15 c. c. strong acid, and boiled to expel excess of NH_3, or until there is but a faint odor of it, and the precipitated iron and aluminum hydrates, after settling, are washed once by decantation and slightly on the filter. Setting aside the filtrate, the precipitate is dissolved in hot dilute HCl, the solution passing into the beaker in which the precipitation was made. The aluminum and iron are then reprecipitated by NH_4OH, boiled and the second precipitate collected and washed on the same filter used in the first instance. The filter paper, with the precipitate, is then placed in a weighed platinum crucible, the paper burned off and the precipitate ignited and finally blasted 5 minutes, with care to prevent reduction, cooled and weighed as $Al_2O_3 + Fe_2O_3$.†

Fe_2O_3: The combined iron and aluminum oxides are fused in a platinum crucible at a very low temperature with about 3 to 4 grams of $KHSO_4$, or, better, $NaHSO_4$, the melt taken up with so much dilute H_2SO_4 that there shall be no less than 5 grams absolute acid and enough water to effect solution on heating. The solution is then evaporated and eventually heated till acid fumes come off copiously. After cooling and redissolving in water the small amount of silica is filtered out, weighed, and corrected by HFl and H_2SO_4.‡ The filtrate is reduced by zinc, preferably by hydrogen sulphide, boiling out the excess of the latter afterwards whilst passing CO_2 through the flask, and titrated with permanganate.‖ The strength of the permanganate solution should not be greater than .0040 gr. Fe_2O_3 per c. c.

CaO: To the combined filtrate from the $Al_2O_3 + Fe_2O_3$ precipitate a few drops of NH_4OH are added, and the solution brought to boiling. To the boiling solution 20 c. c. of a saturated solution of ammonium oxalate is added, and the boiling continued until the precipitated CaC_2O_4 assumes a well-defined granular form. It is then allowed to stand for 20 minutes, or until the precipitate has settled, and then filtered and washed. The precipitate and filter are placed wet in a platinum crucible, and the paper burned off over a small flame of a Bunsen burner. It is then ignited, redissolved in HCl, and the solution made up to 100 c. c. with water. Ammonia is added in slight excess, and the liquid is boiled. If a small amount of Al_2O_3 separates this is filtered out, weighed, and the amount added to that found in the first determination, when greater accuracy is desired. The lime is then reprecipitated by ammonium oxalate, allowed to stand until settled, filtered, and washed,§ weighed as oxide by ignition and blasting in a covered crucible to constant weight, or determined with dilute standard permanganate.**

MgO: The combined filtrates from the calcium precipitates are acidified with HCl, and concentrated on the steam bath to about 150 c. c., 10 c. c. of saturated solution of Na (NH_4) HPO_4 are added, and the solution boiled for several minutes. It is then removed from the flame and cooled by placing the beaker in ice water. After cooling, NH_4OH is added drop by drop with constant stirring until the crystalline ammonium-magnesium ortho-phosphate begins to form, and then in moderate excess, the stirring being continued for several minutes. It is then set aside for several hours in a cool atmosphere and filtered. The precipitate is redissolved in hot dilute HCl, the solution made up to about 100 c. c., 1 c. c. of a saturated solution of Na (NH_4) HPO_4 added, and ammonia drop by drop, with constant stirring until the precipitate is again formed as described and the ammonia is in moderate excess. It is then allowed to stand for about 2 hours when it is filtered on a paper or a Gooch crucible, ignited, cooled and weighed as $Mg_2P_2O_7$.

K_2O and Na_2O: For the precipitation of the alkalies, the well-known method of Prof. J. Lawrence Smith is to be followed, either with or without the addition of $CaCO_3$ with NH_4Cl.

SO_3: One gram of the substance is dissolved in 15 c. c. of HCl, filtered and residue washed thoroughly.*

The solution is made up to 250 c. c. in a beaker and boiled. To the boiling solution 10 c. c. of a saturated solution of $BaCl_2$ is added slowly drop by drop from a pipette and the boiling continued until the precipitate is well formed, or digestion on the steam bath may be substituted for the boiling. It is then set aside over night, or for a few hours, filtered, ignited and weighed as $BaSO_4$.

TOTAL SULPHUR: One gram of the material is weighed out in a large platinum crucible and fused with Na_2CO_3 and a little KNO_3 being careful to avoid contamination from sulphur in the gases from source of heat. This may be done by fitting the crucible in a hole in an asbestos board. The melt is treated in the crucible with boiling water and the liquid poured into a tall narrow beaker and more hot water added until the mass is disintegrated. The solution is then filtered. The filtrate contained in a No. 4 beaker is to be acidulated with HCl and made up to 250 c. c. with distilled water, boiled, the sulphur precipitated as $BaSO_4$ and allowed to stand over night or for a few hours.

LOSS ON IGNITION: Half a gram of cement is to be weighed out in a platinum crucible, placed in a hole in an asbestos board so that about 3/5 of the crucible projects below, and blasted 15 minutes, preferably with an inclined flame. The loss by weight, which is checked by a second blasting of 5 minutes, is the loss on ignition.

May, 1903: Recent investigations have shown that large errors in results are often due to the use of impure distilled water and reagents. The analyst should, therefore, test his distilled water by evaporation and his reagents by appropriate tests before proceeding with his work.

*If anything remains undecomposed it should be separated, fused with a little Na_2CO_3, dissolved and added to the original solution. Of course a small amount of separated non-gelatinous silica is not to be mistaken for undecomposed matter.

*For ordinary control work in the plant laboratory this correction may, perhaps, be neglected; the double evaporation never.

†This precipitate contains TiO_2, P_2O_5, Mn_3O_4.

‡This correction of Al_2O_3 Fe_2O_3 for silica should not be made when the HFl correction of the main silica has been omitted, unless that silica was obtained by only one evaporation and filtration. After two evaporations and filtrations 1 to 2 mg. of SiO_2 are still to be found with the Al_2O_3 Fe_2O_3.

‖In this way only is the influence of titanium to be avoided and a correct result obtained for iron.

§The volume of wash water should not be too large. Vide Hillebrand.

**The accuracy of this method admits of criticism, but its convenience and rapidity demand its insertion.

*Evaporation or dryness is unnecessary, unless gelatinous silica should have separated and should never be performed on a bath heated by gas. Vide Hillebrand.

ANNEALING OVENS 29 ft. 6 ins. × 22 ft. 6 ins. have been built for the Northwestern Malleable Iron Works, of Milwaukee, Wis., and are described in the "Iron Age." Each holds 99 pots, and the castings are almost all of very small size. The arched roof has a radius of 32 ft. 6 ins., and each of the two ovens has about 30 sq. ft. of grate surface. As the ground was not of such a nature as to carry a brick smokestack high enough to give the desired draft pressure, which is 1½ ins. of water, induced draft has been adopted. For this there is a fan 110 ins. diameter, directly connected to the shaft of a vertical inverted engine with cylinder 6 × 6 ins. The company's plant has a capacity for turning out 85 tons of castings per day.

FREIGHT RATES on German railways are double those of American railways, according to the following published extract from a lecture delivered before the Austrian Society of Engineers and Architects by Mr. Franz R. Le Monnier:

The average freight rate of the American railways, in spite of the growing railway trusts, has declined strongly for the last thirty years. Count Moltke, who closely studied these rates, stated that in 1870 the American railways received 5.2 pfennigs (1.2 cts.) per ton per kilometer (0.6214 mile); in 1900, however, the rate was only 1.81 pfennigs (0.43 ct.); therefore the American railways transport their freight at exactly one-half the charge of the German State Railways, which is 3.62 pfennigs (0.86 ct.) per ton per kilometer (0.6214 mile). Where the transportation is partly by rail and partly by vessel, the rate declines still further. The freight charge for wheat over the railways from Chicago to New York is only about 9 marks ($2.14) per ton, while in Germany, upon the railways between Bromberg and the Rhine (only about half the distance), the charge is 38 marks ($9.04)—more than four times as much.

THE NORTHERNMOST RAILWAY LINE in the world will be formally opened some time in July. This line, the Ofoten Railway, runs from the head of the Gulf of Bothnia northerly to the head of Ofoten Fjord, on the northwestern coast of Norway. It traverses valuable iron ranges. Freight has been moved over portions of the line for nearly a year past, and no especial difficulties have been produced by the winter conditions.

A PROCESS FOR MATURING CEMENT (Portland or Rosendale) has recently been patented by Mr. F. W. Cappelen, M. Am. Soc. C. E., late City Engineer of Minneapolis; Mr. H. A. Gerdes, Inspector of Cements for the city of Minneapolis, and Messrs. Eiftman and Cull, Mining Engineers. The process, which is auxiliary to the present methods of manufacture, will enable the maker to put the product of his mill direct from the screens into the cars without ageing it in the warehouse. We have received sample pats of two well known cements from car loads which were condemned due to cracking under the boiling test. We also have sample pats made from the same car loads after treatment by the new process, and they are perfectly sound after boiling.

A CONCRETE-STEEL THREE-HINGED ARCH BRIDGE.

A concrete-steel arch bridge at Laibach, Austria, built to designs of Prof. J. Melan, of Prague, presents some rather unusual features to engineers familiar only with American practice. It is a three-hinged structure, the hinges for the concrete portion being made of separately molded concrete hinge blocks, while the steel reinforcement is independently hinged on pins resting between cast shoes. Each complete hinge is therefore composite, part metal and part concrete. The

FIG 1. CONCRETE-STEEL THREE-HINGED ARCH BRIDGE AT LAIBACH, AUSTRIA.
Designed by Prof. J. Melan, Prague. Architecture by Mr. Zaninovich, Trieste.

provision of hinges is somewhat rare in concrete-steel arches, though it is an approved practice for stone arch bridges. A further feature in which this bridge follows modern practice in stone arches is the use of secondary or relieving arches instead of solid spandrel filling for supporting the roadway over the main arch ring. These secondary arches also have reinforcing steel imbedded, but are not hinged. The following further particulars regarding the structure are taken from an illustrated article by Prof. Melan in the "Zeitschrift des Oesterreichischen Ingenieur und Architekten Vereins:"

The bridge crosses the Laibach River on a skew of 9° 14'; the clear span of the arch is 108.2 ft., while the clear rise is 14.6 ft. These dimensions were selected partly with reference to a future channel improvement, by which the river is to be deepened and confined by retaining walls. In the present condition of the river the high water level is above the springing line of the arch. Unusually deep and wide foundations of concrete on piling were rendered necessary by the poor condition of the subsoil. A special effort was made to secure an aesthetic effect; the structure is faced with ornamental molded concrete slabs, and the molded concrete balustrade is relieved with bronze candelabra and portal figures. The general effect produced is well shown by Fig. 1.

GENERAL DESIGN.—The bridge has a 33 ft. roadway and two sidewalks, each 6½ ft. wide. The outside width of the main arch is about 50 ft. The loading for which the structure was calculated consists of two wagons weighing each 13 tons, and a uniformly distributed live load of 96 lbs. per sq. ft. The axle-loads were assumed distributed each over a width of 8 ft.

The secondary arches, of which there are three each side of the middle, have clear spans of 7.4 ft., and their intrados is semicircular. The main arch has a span of 109.34 ft. c. to c., and a rise, c. to c., of hinges, of 14.34 ft. Its intrados is three-centered, following as nearly as possible the curve of pressure for half load. On the middle 33-ft. section the intrados has a radius of 123, while the radius of the outer portions is 98 ft.

The bridge is shown in longitudinal and transverse section in Figs. 2 and 3. The thickness of the main arch ring varies from 20 ins. at the crown to 27½ ins. at the haunches and 25½ ins. at the skewbacks. Its reinforcement consists of arched steel lattice girders whose flanges are everywhere about 2 ins. within the concrete. These girders, of which there are 14, spaced about 3.3 to 3.8 ft. apart and connected transversely by four sets of steel cross-framing, are parallel to the axis of the bridge, i. e., are skewed with respect to the abutments.

The secondary arches are 6 ins. thick in the center, but thicken rapidly toward the haunches. Their reinforcement consists of curved 4-in. I-beams, which are spaced 3.3 ft. apart. These I-beams, however, are not parallel to the reinforcing girders of the main arch, but lie in vertical planes normal to the abutments.

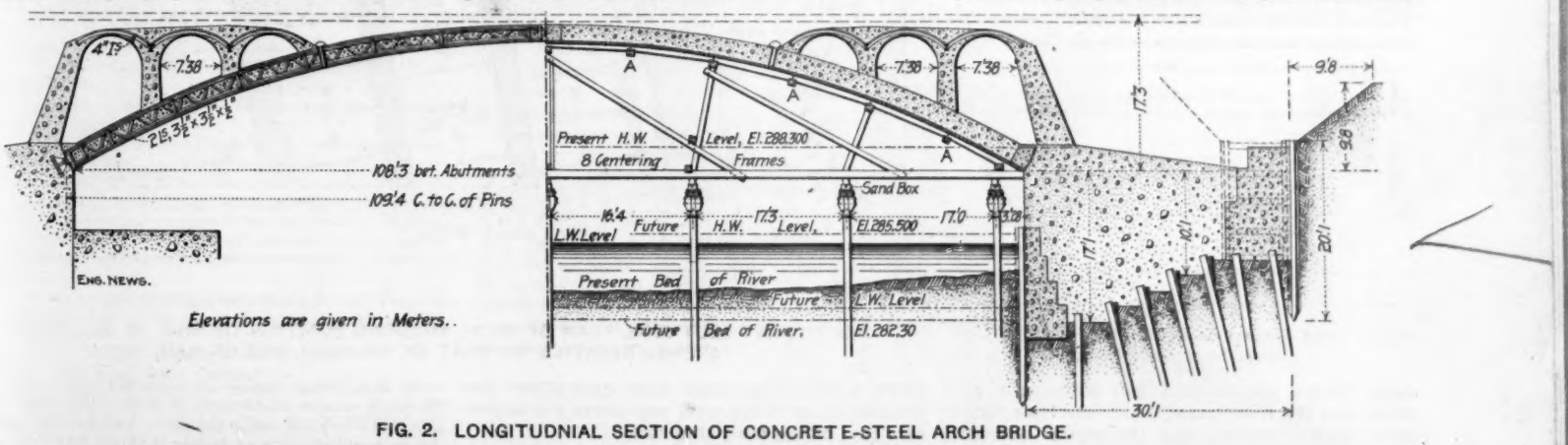


FIG. 2. LONGITUDNIAL SECTION OF CONCRETE-STEEL ARCH BRIDGE.

Among the details of the structure the hinges are interesting. Figs. 4 and 5 show respectively the abutment hinges and the center hinge. The concrete hinge blocks were molded several months before the erection was bgun. Full and uniform contact of the curved abutting faces was secured by placing in the joint a strip of hard lead, 4 ins. wide by 1/32-in. thick. The center hinge of the steel ribs is a simple abutting pin-joint, as will be seen from the drawing, but the skewback hinges of the ribs have wedges for accurate adjustment of the span.

After the arch was completed and the centering removed, the hinges of the metal ribs were encased in concrete. As all the permanent deflection of the arch had probably taken place before that time, it was thought that this concreting would not interfere with the proper action of the hinges; as a matter of fact no cracks have appeared here since the bridge has been put into service.

The construction of the roadway and sidewalks is sufficiently shown in Fig. 3; it may be added that the asphalt is confined at each end of the bridge by a transverse curb of granite blocks. A gas main and an electric cable are carried over the bridge in separate conduits (not shown in the drawing) formed in the concrete under the sidewalks. The back of the main arch and also the upper surface of the secondary arches were covered with a 1-in. layer of asphalt, to shed water, and drain holes were formed through the arch ring at all points of depression.

CONCRETING AND ERECTION.—In the concrete work and the erection some novel points are to be noted. The fact that extra large foundations would be required on account of the nature of the ground was learned only after the contract had been let. To economize in the cost of these foundations, their lower portions were made of a concrete mixed 1 part of cement to 14 parts of washed sand and gravel, a rather unusually meager mixture. The proportion of cement was increased gradually toward the top of the foundation, to a mixture of 1 part of cement to 8 of sand and gravel (1:3:5). The concrete blocks for the hinges, which, as already noted, were made before the work commenced, are composed of 1 part of Portland cement to 4 parts of washed sand and broken stone. The concrete of the arch ring was mixed 1 part of cement, 3 parts of washed sand and 3 parts of stone, the latter being half gravel and half broken stone.

The falsework used to support the arch during its construction is shown in Fig. 2. Eight centering frames were used, spaced about 7 ft. apart and braced together by diagonal battens. Each frame rested on seven supports, consisting of single piles; on the top of each support was fixed a sand box. The upper members of the centering frames were not continuous, but were cut in the middle of each panel and here rested on cross timbers hung from the steel reinforcing ribs by means of hangers, as shown in Fig. 6. By this means about half the total dead weight during erection was carried by the steel arches and half by the centering.

After the hinge-blocks were placed, the concreting of the main arch proceeded from four points, being begun simultaneously on each side of the center and at each skewback. The steel reinforcing arches were coated all over with a cement wash, and during concreting great care was taken to pack the concrete closely around the steelwork. Concreting was carried on uniformly along the full width of the arch, and at the end of each day's work a radial partition was placed against the fresh concrete. Before beginning work the next morning the radial face thus formed was roughed up with picks and coated with a mortar of 1 part cement to 3 parts sand, before fresh concrete was placed against it; this is thought to

have secured a thorough joint between the sections placed on successive days.

The concreting of the main arch ring occupied ten days; seven weeks later the centers were lowered, the secondary arches having also been completed by that time.

The top of the roadway was designed to be level, longitudinally. The centering was adjusted so as to have an equivalent camber of 6 ins., but the total deflection under the weight of the completed arch was only about 2 ins.; when the centering was dropped the crown of the arch lowered only about ⅛-in., so that about 4 ins. of camber remained.

In dropping the centering an unexpected circumstance occurred. The sand boxes had been filled with carefully-dried fine sand, but high water during the erection had submerged the boxes, and the river water, which carried considerable clayey matter, had thoroughly wet the sand. When it came to lowering the centers the sand would not run out of the boxes, and it was necessary to scrape it out, and as a result the lowering proceeded by irregular jerks. However, no damage to the arch resulted from this.

ARCHITECTURAL FINISH.—The appearance of the completed bridge is well reproduced in the view, Fig. 1. It will be observed that the spaces between the secondary arches are filled out by decorative panels and scroll work. It was at first contemplated to make this work of bronze or forged iron, but to keep down the cost it was later decided to use molded plates of concrete. The molded face of the main arch ring and of the abutments was produced in like manner, the concrete plates being placed and jointed like stone masonry. The balustrade is similarly made of molded concrete. Those parts of the façade where molded blocks were not required were provided with a facing 4 ins. thick of 1 part cement to 4 parts limestone crushed to the fineness of sand; this facing was put in place while the arch concreting was in progress, so that it forms an integral part of the structure, and after completion of the work was bush-hammered to give it a smooth surface.

The bridge was opened for traffic in October, 1901, fourteen months after breaking ground for the foundations. The weight of metal in the bridge is about 67 tons, of which about 16 tons are in the hinge castings. The total cost of the bridge was about $32,000, of which the metal work cost about $6,000, and the ornamental work about $2,000. The expensive foundation work represents a considerable part of the remaining cost.

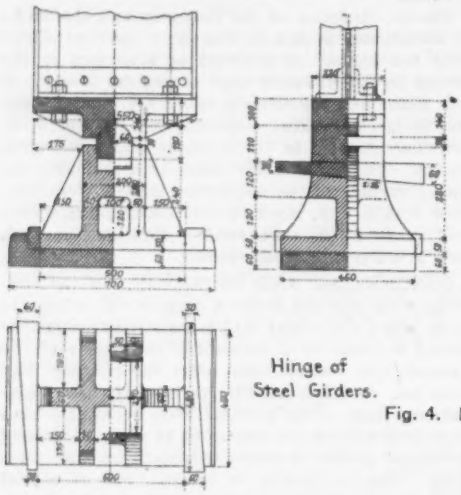


Fig. 4. DETAILS OF SKEWBACK HINGE.

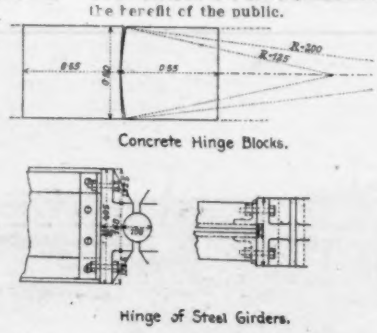


Fig. 5. Details of Center Hinge.

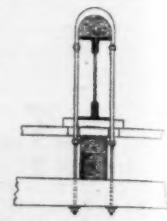


Fig. 6. Method of Hanging Centering to Steel Reinforcing Ribs.

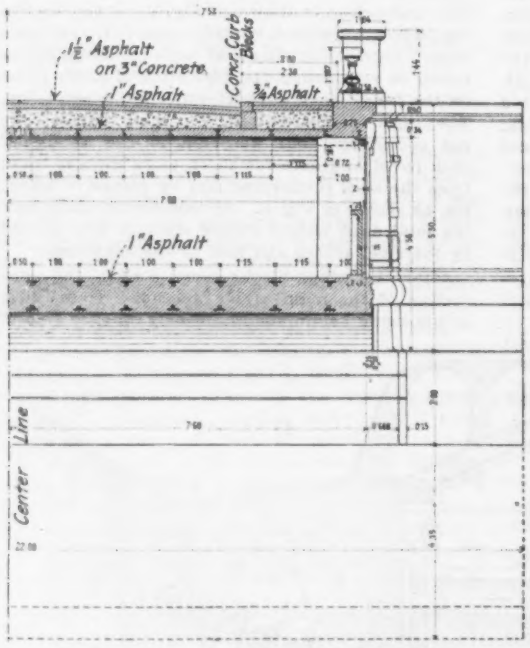


Fig. 3. Half Transverse Section of Haunch of Concrete-Steel Arch Bridge.

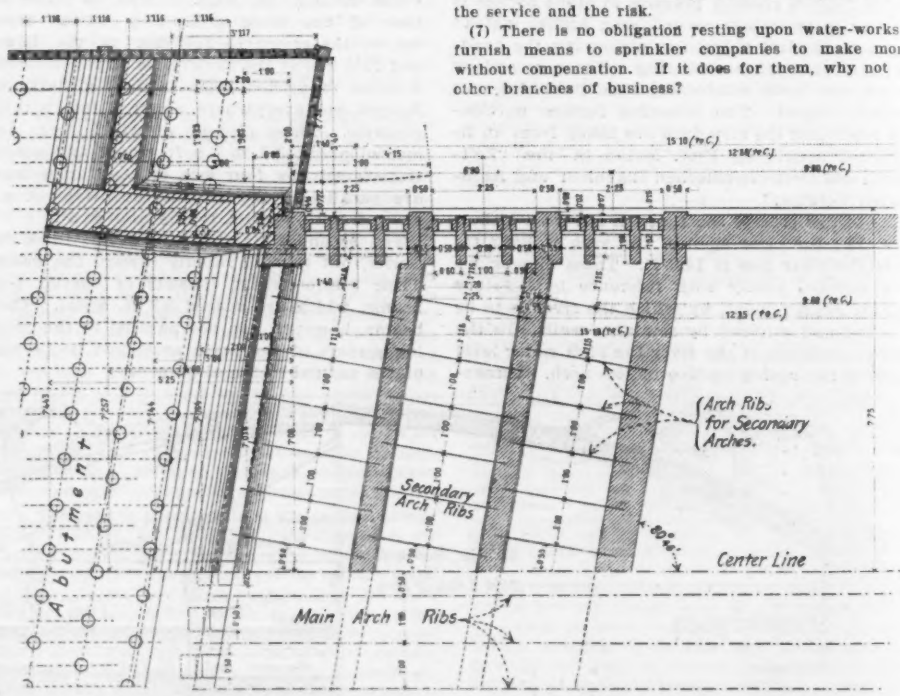


FIG. 7. PART SECTIONAL PLAN OF ARCH, SHOWING POSITION OF RIBS OF SECONDARY ARCHES RELATIVE TO THAT OF TRUSSED RIBS OF MAIN ARCH.

SHALL WATER DEPARTMENTS CHARGE FOR PRIVATE FIRE LINES, AND SHALL METERS BE PLACED THEREON?*

By F. A. W. Davis.†

There are the questions that have often been propounded in the last year, and by request of the Secretary I have prepared the following paper in reply, with a feeling that the subject has been thoroughly discussed by able members of the three water-works associations and the following conclusions reached:

(1) No city or corporation should furnish a valuable service without value received.

(2) Water on the inside of a building ready for an emergency in case of a fire at any time is a valuable service.

(3) It is a special service to a property-holder or manufacturer for the purpose of reducing cost of insurance and reducing the risk from fire, and does not stand in the same relation to the water-works as fire hydrants which are paid for directly or indirectly and which are for the benefit of the public.

(4) The saving in insurance rates, which could not be possible otherwise, justifies a reasonable compensation for the service and is the same principle that justifies a man in buying labor-saving machinery for which he is willing to pay.

(5) There is no reasonable ground for asking that the property of any water-works should be placed at the convenience and for the special protection of any one without compensation.

(6) No water-works is justified in running large mains into a building for the extinguishment of fires, which if they broke would jeopardize surrounding property and the whole water-works machinery, without compensation for the service and the risk.

(7) There is no obligation resting upon water-works to furnish means to sprinkler companies to make money without compensation. If it does for them, why not for other branches of business?

(8) If sprinkler services are permitted by a city or a company, careful consideration must be given to the system, machinery and size of the mains, and provisions made for extraordinary demand in case of an accident.

(9) Often these sprinkler attachments add to the cost of the system by requiring larger mains.

(10) The annual loss to water departments and companies by reason of the water being taken from fire lines without pay or knowledge or consent, based upon the experience of Brooklyn, Cleveland and Chicago, must be many thousands of dollars annually.

(11) The objection to meters by the insurance companies is valid, for the reason that they reduce the pressure and there is a possibility that they may stop at a critical time.

(12) The objection from a water-works standpoint: The cost of meters, $250; interest on same at 6%, $15 a year.

*A paper read before the annual convention of the American Water-Works Association, held at Detroit, Mich., June 23-26, 1903.

†Vice-President Indianapolis Water Co., Indianapolis, Ind.

addition to this, the cost of maintenance and examinations.

(13) The facilities afforded for getting water through the meter without its registering, which can be done when the current is not great enough to cause a 4-in. meter to register.

(14) Arguments of the sprinkler company are that the system saves hose and water, and by putting out fires in the beginning it lessens the risk to adjoining property. (Though they are not always successful in putting out the fires in the beginning.) Admitting for the sake of argument that they are successful, does this success justify the water-works in furnishing water to do this without compensation, besides taking the moral and physical hazards pointed out?

(15) It is claimed by the sprinkler companies and the insurer that the cost of the installation of the sprinkler system is large and must be considered; besides, the sprinkler system prevents fires developing into a general conflagration and thereby becomes to a certain extent protection to other property, and for these reasons there should be no charge. These statements should receive consideration, but it must be borne in mind that the sprinkler companies make a profit (which is right) out of the construction of the system and the insurer gets such a reduction of rates that his equipment is paid for in two or three years, and thereafter a large saving is made annually in the cost of his insurance. It is wholly immaterial to the water department if the one make a profit and the other a saving, but it is important that the water department receives compensation for its property and services, which are absolutely essential to the accomplishment of the purpose for which the sprinkler system is installed.

(16) If a water department or company furnishes facilities for the extinguishment of fires to one without compensation, why not to all, including private residences? Surely a right or privilege should not be granted to one and refused to another.

(17) As a matter of equity there should be a reasonable charge for the services, and the sprinkler companies should construct their systems with devices which will present water being drawn from fire protection lines, except in case of fire. This would do away with the necessity for meters.

The committee of the New England Water-Works Association has reported that there should be a reasonable charge; also the Central States and American Water-Works Associations; so it would seem there remains nothing to be said on the subject, except to discuss the advisability of using meters and rates. If the sprinkling companies will furnish devices which will do away with meters, that would leave the rate question to be settled by local conditions, as well as the size of opening in the mains.

INSURANCE SAVED.—Estimated reduction in insurance rates to 14 business concerns in the City of Indianapolis, by reason of inside fire protection, per annum, $44,650. Cost per annum for the protection afforded by the water company, $950. for all.

BREAKAGE OF MAINS.—The breaking of mains in buildings having inside fire protection lines during fires, resulting in reduction of pressure, occurred in Columbus, Ind.; Knoxville, Tenn., and Champaign, Ill. These few cities are only mentioned to show that the breaking of mains in buildings has occurred, and will occur again, and that they reduce the pressure so that it is of little value.

If I am correctly informed, the courts of Kentucky and New York have held the water-works responsible for lack of pressure, and the Water-Works Company of Louisville, Ky., has been sued for $96,000 for the same cause by the insurance companies.

It will naturally occur to the average water-works man if the water-works are liable, as has been held by the courts, can he afford to and has he a right to put in fire protection lines of a private character which may break, and thereby increase the hazard and the liability for damages, and which may be such an amount as to be serious financially for the owner, whether it is the city or a company.

INDIANAPOLIS CONTRACTS.—Contract and rates of the Indianapolis Water Co. for fire protection lines are as follows:

ATTACHMENT No. .....

To the Indianapolis Water Company:

The undersigned hereby proposes, that if said Water Company will, from its water main in ..... St., in the City of Indianapolis, Ind., extend into the premises No. ..... St., in said City, a ....-inch pipe for the purpose only of furnishing water for fire protection within said building, and will through such ....-inch pipe, furnish water for such purposes, ... he ..... will at once pay said Company the cost of said pipe and the expense of laying it, and as soon as said pipe is laid will connect the same with a pipe in said building which is to be used only for fire protection purposes in said building; and for the water supplied by said company in such ....-inch pipe, the undersigned hereby agree..... to pay said Company the sum of ..... dollars per year, payable quarterly, in advance, without relief from valuation or appraisement laws.

And the undersigned agree.... to comply with all the rules and regulations of said Water Company now in force or hereafter made, and to give free access to the premises aforesaid to the employees of said company at all reasonable hours for the purpose of inspecting the pipes and openings, sealing the valves, setting meters in connection with said pipes to be used for fire protection purposes.

The undersigned further agree.. that no water shall be taken from or through said fire protection pipe except for the extinguishment of fires, and that no extension, alteration or opening shall be put in said pipe, without the consent of said Water Company first obtained.

The undersigned further agree.... that upon any breach by the undersigned of any of the agreements herein expressed, the said Water Company may, at its election, and without notice, at once terminate this contract, and shut off and permanently discontinue the supply of water through said fire protection pipe, and retain all moneys theretofore paid to it by the undersigned, and waiver as to any breach shall not operate an estoppel as to subsequent breaches; and, also, that said Water Company may, at its election, after thirty days' notice in writing to the occupant of said premises and the refunding of any money then in its hands for the unexpired part of the then current quarter, terminate this contract and discontinue the supply of water through said pipe so to be laid by it.

And it is expressly understood and agreed, that by its acceptance of this proposition, said Water Company shall not be held to guarantee the sufficiency of its water supply for fire protection in said buildings; and that said Water Company shall be in no way liable to any one for a failure of the water supply through said fire protection pipes, whether occasioned by the scarcity of water, or the breaking of any pipe, valve or connection, in said Company's mains, or from any other cause whatsoever, nor shall said Water Company be liable in any way to any one for the consequences of such failure of supply of water, however occasioned.

.....

FIRE PROTECTION LINES.

Under special contract, 4-in. pipe running into single building, $50 per annum; 3-in., $35; 2-in., $25.

No fire lines run into buildings or fire lines supplied with water unless water is used for general purposes.

No charge for cutting main, for valve or for connection.

A HIGH MASONRY DAM IN GERMANY.

An unusually high masonry dam, shown in the two drawings reproduced herewith, is being built across the small river Urft, near the city of Aachen, in the western part of Rhenish Prussia, Germany. It has the object of equalizing the flow of the stream, to furnish a dry-weather supply for agriculture as well as to prevent the annual damage from floods in this river and its confluent,

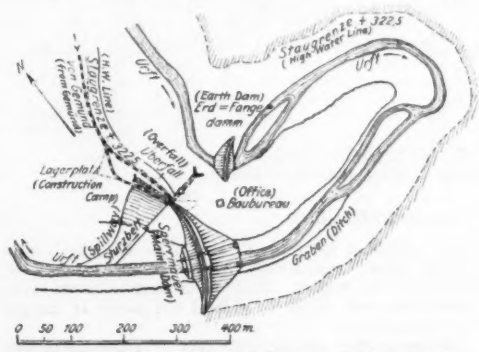


Fig. 1. Location Plan of Dam and Temporary Embankment.

the Ruhr. This dam, with a height of 190 ft. from foundation to top, will form a reservoir 174 ft. in greatest depth, capable of holding 1,600,000,000 cu. ft. of water. We take our illustrations, together with a very brief description of the structure, from the "Zeitschrift des Vereins Deutscher Ingénieure."

The location of the dam is shown in the map Fig. 1, while its maximum section is given in Fig. 2. The structure, which is about 750 ft. long, is arched upstream on a radius of about 650 ft. It is 18 ft. wide on top, 3 ft. above high-water level. The cross-section is roughly triangular, with an upstream face that is vertical in its lower portion, but is battered back considerably in its upper half.

Against this upstream face is an earth fill extending to over half the height of the dam, with its upper slope (1:2) paved. The material of the dam itself is argillaceous slate, laid in courses inclined against the line of pressure. On the upstream side the masonry has a 3-ft. facewall of trap rock, stepped, as shown in Fig. 2, on the battered portion. Between the facewall and the body masonry is a 1-in. layer of cement, coated with asphalt. This is intended to prevent seepage of water into the masonry, but for the case that such seepage does occur there are provided two rows of drain pipes set vertically in the masonry, about 8 ft. apart in the rows. The pipes of each row (2½-in. clay pipe) are connected at the bottom by a 6-in. header leading to the drain tunnels.

There are two of these drain tunnels, running through the dam near its center (as shown in the plan Fig. 1), where they tap the reservoir at its lowest level. They are each closed by a gate-chamber at the upstream face of the masonry, and continue thence through the earth fill as masonry tunnels. From each gate-chamber rises a gate-tower, through which pass the valve-stems for operating the stop-gates. Through the body of the dam, i. e., downstream from the gate-chambers, the drain tunnels do not carry the water directly, but serve each merely as a conduit for a 24-in. steel blow-off pipe. This pipe extends into the gate-chamber and there has two gate-valves; another gate-valve in the pipe is provided just below the gate-chamber, so as to be accessible from the drain tunnel.

An overfall weir about 300 ft. long is built at the northerly end of the dam. The crest of the weir, instead of being in a straight line, is scalloped or wave-shaped in plan, with panels 20—25 ft. wide separated by buttresses. Several of the panels are fitted with gates, to increase the discharge capacity of the weir. The wasteway below the weir is in the natural rock, which is cut into steps 5 ft. high to break the fall of the water; an 18-in. facing of concrete on these steps protects the rock from erosion.

For the construction of the dam the water of the Urft is held back by an earth embankment thrown across the river some distance above the building site, where a loop in the river valley leaves only a narrow rock ridge between the locations of the dam and of the temporary embankment. A tunnel through this ridge carries off the water during construction, but this tunnel, which passes under the spillway of the new dam, is to be provided with a gate to serve as a drain tunnel when the permanent work is completed.

The water stored in the reservoir is to be used in part as a source of power. About a mile north of the dam a tunnel 9,200 ft. long has been built, which leads from the reservoir into an adjacent valley, where it supplies water under 360 ft. head to a power house. Here eight turbines of 1,250 HP. each will drive electric generators for transmission to surrounding towns. The entire work is being done under the direction of Prof. Otto Intze, of Aachen, and according to his plans. The contractors for the work are the firm of Philip Holzmann & Co., of Frankfurt.

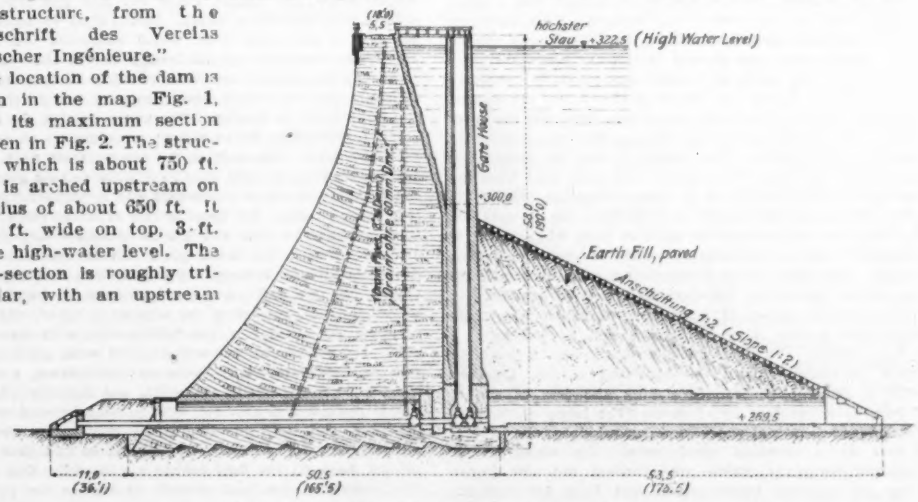


FIG. 2. MAXIMUM SECTION OF DAM.

THE ELECTRICAL EQUIPMENT OF A GOLD DREDGE.*

By Ralph L. Montague, Assoc. M. Am. Inst. E. E.†

Before describing the electrical equipment, it would be well, perhaps, to say a few words relating to the use and object of a gold dredge.

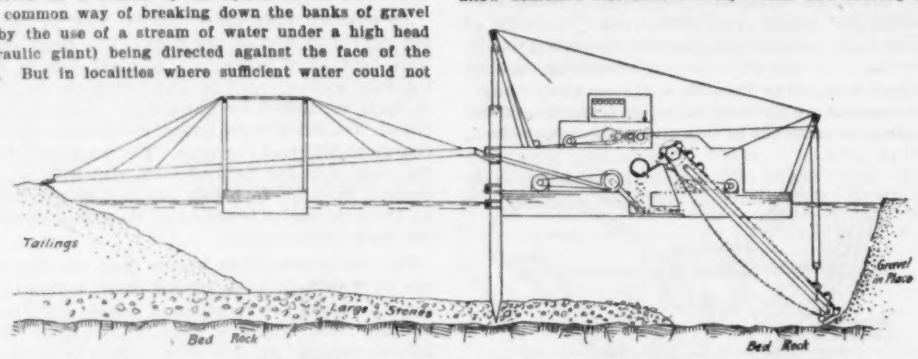
The occurrence of gold in deposits of gravel is caused by the action of some river or glacier which in its course has broken up and carried along in the form of gravel the rock that formed the boundary of its channel.

When gold-bearing veins exist in this rock, the gold in the course of time becomes freed from its surrounding ore, and is, together with the gravel already formed, deposited at some point where the velocity of the water ceases to be high enough to keep the gravel in motion.

Placer mining is the name by which the process of extracting the gold thus deposited from its surrounding gravel is known. Briefly, it consists of feeding gravel and water into a rectangular trough (sluice box), set on a grade, on the bottom of which depressions or hollow spaces (riffles) are provided. The gold, owing to its high specific gravity, settles in these depressions, while the gravel is carried on down the sluice and dumped at the end.

To accomplish this process on a large scale there must be sufficient grade to the deposit of gravel and its underlying bedrock to prevent the worked-out gravel (tailings) from interfering with the excavating operations which are feeding the gravel into the sluice.

Where the natural grade is not sufficient for this purpose a grade must be produced by artificial means. Until dredging was applied for this purpose the method known as hydraulic mining was used. It consisted of sinking a pit to bedrock, the gravel being lifted into a sluice box supported on a trestle by an hydraulic elevator. The most common way of breaking down the banks of gravel was by the use of a stream of water under a high head (hydraulic giant) being directed against the face of the bank. But in localities where sufficient water could not



TYPE OF ELECTRIC GOLD DREDGE.

be obtained for hydraulic mining, except at a prohibitive expense, the deposits have not been considered of any value until the advent of the gold dredge. Furthermore, where sufficient water was available to work the deposit by the hydraulic method the same water if applied to turbines for the generation of electric power to be used to drive dredges would handle from two to two and a half times more gravel.

Fig. 1 gives in outline a type of gold dredge. The excavating is done by the endless chain of buckets, which revolving round the upper tumbler dumps the gravel into a revolving screen (termed a grizzly). Jets of water play on the gravel and thoroughly wash it; the fine gravel and gold fall through the openings in the grizzly into a sump, from which it is picked up, together with sufficient water and elevated into the sluice box. The coarse gravel, large stones, etc., are allowed to fall overboard, being of no value. The gold, as already described, is recovered in the riffles forming the bottom of the sluice; the washed gravel, carrying practically no values, falls over the lower end of the sluice and fills up the cut that was previously made by the dredge. The dredge is held in position by the anchor or spud at the stern, and side lines from the bow permit the buckets to be freed across the face of the cut. There are two spuds on a dredge; one is used for holding the dredge and the other is used when a cut is completed and it becomes necessary to step the dredge ahead. With this type of dredge it is possible to handle .85 cu. yds. of gravel per HP.-hour delivered on board.

The various pieces of machinery, together with their respective motors, may be summarized as follows:

The endless chain of buckets driven by a variable speed reversible motor; the revolving screen, together with a small pump for furnishing the jets of water, driven by a constant speed motor; the large centrifugal pump for elevating water and gravel into the sluice, driven by a constant speed motor; the winch which revolves drums to which are attached the side lines; lines for hoisting bucket chain and lines for hoisting spuds; is driven by a variable speed-reversible motor. The first dredge that made a commercial success as a placer mining machine in the United States was installed at Bannack, Mont., in 1895. The property owned by the company (The Gold Dredging Co.) included a water right and ditch, which if extended about two miles would give a head of 340 ft. As the water at this head would give sufficient power to drive a dredge it was decided to use electric power in preference to steam. The fuel cost was for wood $4.50 per cord and for coal $12 per ton. The writer installed the electric equipment, which consisted of a 100-KW., 500-volts, d.-c. generator, running at 650 revs. per min., and four street railway motors.

The generator was direct connected to a 26-in. Pelton water wheel, by means of an insulated coupling. The power was transmitted over .000 stranded conductors, triple braid, weather proof, from the pole line on shore to a reel on the dredge. Two of the motors were mounted on wooden bases sliding on rails and had overhung pulleys in place of the pinions; they were used to operate the winch. One motor drove the drums for hoisting the spuds and the other motor operated the balance of the winch. The controllers were of the rheostatic type. The other two motors were geared to counter shafts having pulleys mounted on them, the whole mounted on wooden bases, and they were controlled by d.-p. switches and starting resistances of iron wire; one of these motors was supplied with a d.-p. reversing switch. This motor had to drive: (1) the endless chain of excavating buckets, (2) the revolving screen, and (3) a secondary chain of buckets which elevated the screened gravel into the sluice. The other motor drove a 10-in. centrifugal pump that supplied water to the sluice box.

No trouble was encountered with the generating end of the plant, with the exception of a little difficulty owing to a mistake in the water-wheel design, which necessitated the lengthening of the water-wheel shaft, so as to allow sufficient clearance between wheel and housing to permit the dead water to fall away from the wheel; but, on the dredge, considerable trouble was experienced in operating the motors. Both these motors were overloaded and their design in general left plenty of margin for improvement.

During the first season (1895) very little excavating was done, but enough work was done to demonstrate that these motors would not give satisfaction if the load was not lightened on them; accordingly, during the interval between the seasons of 1895 and 1896, the motor driving the 10-in. centrifugal pump was replaced by a shunt wound motor and the load of the other railway motor was divided between it and the motor taken off the pump. One motor drove the bucket chain (1) and the other, the grizzly and secondary chain. Mechanical changes were effected at the same time, which permitted the almost continuous operation of the dredge during the season of 1896. As the motors now were running on an average of 23 hours per day, there were plenty of opportunities for the weak spots to develop. The motor driving the bucket chain (excavating) burnt out an armature on an average every 30 days. The ends of the commutators next to the bearing on these motors were constantly getting grounded, the brushholders were overheated and brushholder springs lost their vitality. By keeping two or three extra armatures on hand no time was lost on armature account; but on one occasion the lower yield coil of the motor that drove the grizzly became badly grounded, and short-circuited. As we had no spare field coils on hand I unwound the coil, rewinding the wire on a barrel, and after painting the interior of the field bobbin with insulating paint, I rewound the coil with the old wire, painting the wire as it was laid on. From my recollection, I should say the wire was about .000 solid, and anybody who has had to rewind a magnet coil with previously used wire of this size and with practically no appliances, such as sheaves for wire to run over, need not be told that I did not get back on the field-bobbin all the wire that came off; however, I got back enough to answer the purpose and within ten hours from the time of the accident the dredge was in operation again, and the motor lasted to the end of that season.

During the winter of 1896-1897 it was decided to discontinue the use of the secondary chain of buckets for elevating the screened gravel into the sluice, and to use a centrifugal pump of sufficient size to lift this gravel and the necessary water for sluicing it. The new arrangement presented an outline similar to that in Fig. 1.

The experience met with in this the first successful application of a dredge to placer mining in America have been described at considerable length, because this plant was the forerunner of numerous others in localities throughout the mining districts of North and South America, which have created a new demand for labor and opened a field for the investment of millions of dollars.*

I will now endeavor to describe the nature of the loads on the various motors. That on the motor that drives the bucket chain is a peculiar one; it may be described as a combination of an elevator load and a street-car load, but with many less stops and no particular need for very rapid acceleration. When beginning a cut at the surface, the incline up which the buckets travel may be only 20° from the horizontal, the soil as a rule is very loose and the starting torque is comparatively speaking nominal. The motor has only to overcome the inertia due to the mass and weight of bucket chain, viz., from 250 to 400 lbs. per ft. of length. As the buckets become loaded with gravel the load increases. The speed of the bucket chain at the top of the cut can be as high as 60 ft. per minute with safety. This load is similar to an elevator load and is steady; the only fluctuations being caused by a bucket as it revolves round the lower tumbler, taking an extra large bite out of the gravel or missing the gravel and merely bailing water. As the depth of the cut increases, the load increases on the motor because the angle between incline of chain and horizon is greater; and if a series wound direct-current motor is used some resistance should be cut out, if it is still safe to maintain the high speed. When a certain depth is reached and the gravel becomes coarser, is packed harder and large boulders are liable to be met with the speed should be reduced by cutting in resistance; finally, when close to bedrock, the speed should be still further reduced; the buckets should not now exceed a speed of 40 ft. a minute; and in actually scraping bedrock the speed should be still further reduced. The buckets are now nearly empty of gravel; the incline is about 45° with the horizon, and it will be necessary to cut in all available resistance into the circuit of a direct-current series or into the secondary of an induction motor. The construction of this resistance should be such as to permit its being kept continuously in the circuit, as in some fields it is necessary to keep up the scraping of bedrock for several hours because the majority of the gold values are obtained on or near bedrock.

During this, the most important part of the excavation of a cut, the motor is running with a low efficiency; and it may be asked why a two-speed gearing should not be introduced so that the motor would operate efficiently at the low bucket speed. In one case, to my knowledge, a two-speed gear was tried, but finally discarded, because it did not give such general satisfaction as the arrangement at present in almost universal use, viz., a direct-current series-wound motor with rheostatic control or an induction motor with a variable resistance in secondary or rotor circuit.

I have not yet mentioned the condition when the most severe load is thrown on the motor, that is when the motor is stalled. At some point in the train of gears that reduces the high speed of the motor to the low speed of the top or driving tumbler (2.75 — 5 revs. per min.) an adjustable slipping friction is placed and set so that it will slip before the circuit breaker or fuses open the motor circuit. The buckets may encounter an extra large boulder, a submerged timber or stump or the bank may cave down and completely bury them. Let us suppose the buckets are running at a speed of 45 ft. per minute when an obstruction is met that causes the friction to slip. The motor is stopped and when it has come to rest it should be allowed to run back so as to let a little slack into the bucket chain. Sometimes it is necessary to reverse the bucket chain one bucket or link length. On restarting the motor the obstruction may be removed, sometimes, however, it is necessary to stop and start a number of times before this is accomplished; it may be necessary to tighten the friction or hold the circuit breaker from opening, but whatever the nature of the obstruction, it must be overcome because if boulders are not dug out and disposed of they will be liable to form an effective barrier between the buckets and bedrock, which will mean that a large proportion of the gold contained in the cut is not recovered.

Some times it is necessary to stop the buckets, which may be running at full speed, in two or three seconds of time; the only way to do this is to reverse the motor. Therefore, it will be seen that the motor that drives the bucket chain should be capable of withstanding considerable strains, both mechanical and electrical. I have seen a direct-current series-wound motor (500 volts) the normal current of which was 40-60 amperes, take over 200 amperes, while picking up a particularly large boulder; and a 3-phase, 1,000-volt induction motor, normal load 25 amperes per phase with secondary short-circuited.

*Slightly condensed from a paper presented at the 20th Annual Convention of the American Institute of Electrical Engineers, Niagara Falls, N. Y., July 2, 1903.

†Continental Gold Dredging Co., Oroville, Cal.

*At this point (Oroville, Cal.,) there are in operation and building over 20 dredges; each dredge has cost at least $50,000; and the ground to be mined previously worth from $2 to $10 per acre, sells now as high as $1,500 per acre.

has taken 95 amperes per phase, while the buckets were being released from an extra large cave-down of gravel.

The periods of these abnormal loads are limited, rarely exceeding a full minute. While not attempting to deduce an exact formula for determining the necessary rated horse-power of a motor for this class of work, I will state that calculations can be based on the fact that a 50-HP. motor should be installed to drive a chain composed of alternate buckets and links, the bucket capacity being 6 cu. ft. each, built of good mechanical proportions and capable of reaching a maximum depth below water level of 35 ft., the top or driving tumbler being suspended about 14 ft. above water level. In actual work the incline up which the buckets travel does not exceed an angle of 45° with the horizon.

Passing on to the motor that drives the grizzly and small centrifugal pump, very little need be said about it; the load and speed are constant, and in starting the load is light as the centrifugal pump does not begin to pick up water until nearly full speed is reached. A shunt direct-current or induction motor, squirrel-cage type, of 30 HP., is generally used for this load, i. e., where the grizzly and pump are proportioned to screen and wash the gravel excavated by the 5-ft. bucket chain.

The same remarks about load and power required to start apply to the motor that drives the large (12-in.) centrifugal pump that elevates water and gravel into the sluice; the horse-power of the motor can be found from the following formula:

$$\text{HP. of motor} = \frac{(\text{diam. of discharge in inches})^2 \times \text{ft. lift}}{2 \times 20}.$$

A little difficulty may be experienced in finding the exact diameter of motor pulley. There is a certain critical speed at which to run a centrifugal pump that lifts water and gravel; a reduction of 10% below this speed reduces the velocity of the water to a point where it is unable to pick up gravel; 10% above the critical speed calls for an increase of power out of all proportion to the increase of speed of the pump, besides which the resultant velocity of the water in the sluice box may be so high as to prevent the gold from being deposited in the riffles.

There are some peculiar facts concerning the operation of centrifugal pumps by constant speed motors. If such a centrifugal pump is lifting water to a certain height and the motor is fully loaded, a reduction of the head increases the load on the motor and vice versa. This statement may seem to be contrary to all laws regarding the conservation of energy; but the peripheral speed of a centrifugal pump is in proportion to the square root of the height to which the water has to be lifted;* furthermore, the volume of water lifted is not in direct proportion to the speed. If x be the speed of a pump (running efficiently) and y = power used to elevate 1,000 gallons at any head; if x is reduced to x/2, y = 0 (not taking into consideration the losses due to friction), and no water is moved. When the speed is increased above x, the amount of water lifted does not keep up in proportion to increased speed. There is more slip in the water moved by the runner or piston and the efficiency begins to fall off enormously therefore, if we consider the original statement, we see that with a motor fully loaded and pump running at an efficient speed for a head of y feet; reducing y while moving more water, we do so at a very much lower efficiency and consequently overload the motor, provided the speed remains nearly constant.

The last motor to be considered is that which drives the winch used for swinging the dredge, hoisting bucket-chain, etc. The heaviest load this motor has is when the bucket-chain is being raised, the nature of which is similar to that thrown on an elevator motor. In raising the spuds or anchors, there is always some slack in the lines and full torque is not required in starting. In pulling on the side lines very little power is called for, but the speed variations required are beyond the scope of any motor. For instance, in digging off the top soil, the speed of the buckets across the face of the cut may be 20 ft. a minute or more; when bedrock is reached, the speed may be as low as 5 ft. a minute. When moving ahead and the buckets are not touching the gravel, the speed should be as high as possible, viz., 100 ft. a minute or more. In general, the motor is geared to the winch, so as to produce the maximum speed, and the variations are produced by constantly starting and stopping the flow of current to the motor; this may seem to be hard on the controller, but the modern types of controller seem to stand this work remarkably well.

In some more recent winches a two-speed gearing has been adopted and this makes it easier for the controller. The only operation that it is necessary to consider when determining the maximum horse-power of the winch motor is the hoisting of the bucket chain and the speed at which it is to be hoisted. All other loads that may be thrown on this motor are less than this load.

At Bannack, Mont., the writer designed a reel that was placed on the upper deck and the wires (.000 stranded weather-proof insulation) ran over hardwood sheaves and then swung onto the pole line. This class of wire had to be kept out of the water and off the ground. The object of the reel was to make it possible to take up slack or pay it out without stopping the operation of the dredge. This reel was exposed to the weather, but no leakage was ever apparent.

The voltage of the motors should not exceed 400 a. c. or 500 d. c. There is no advantage to be gained in high voltage, as the leads from switchboard to motor never exceed 100 ft. in length. The controllers for the bucket-drive motor and winch motor are located in the pilot house; the auto-starters for the other motors should be placed near the motors and the machinery they control. There should also be a complete system of push buttons and signal bells. Incandescent lamps are used almost exclusively; arc lamps give trouble owing to the constant vibration of the dredge.

For every dredge similar to the one shown in the illustration, the general dimensions may be described as being a 5-ft. open-connected chain dredge capable of excavating to a depth of 35 ft., with a 12-in. centrifugal pump for elevating purposes, 135-KW. should be available at the bus bars on board the dredge for 24 hours per day for direct-current plants, or 170-K. V. A. at .80 power factor where alternating-current induction motors are used. The current is taken aboard in some places at 4,000 volts. I believe about 5,000 volts will be the limit for a few years anyway. The motors and resistances should be such that they will be able to run continuously, without the least danger of overheating. The average running time on the Banack dredge was 23½ hours out of 24. In localities where the mining season is short, owing to severe winters, the usual practice is to run every day and night, Sundays and holidays included, the machinery being overhauled during the winter months.

The mechanical end of the plant causes quite as many shut-down as are desirable and there should be absolutely no loss of time on account of defective electrical apparatus; therefore everything connected with the electrical part of the machinery on the dredge should be as near "fool-proof" as possible.

The ambition of every manager of a dredge is to keep the machinery at work all the time. When we realize that in some fields the possible mining season only lasts 150 days in the year, the value of every minute during this time becomes apparent to the most casual observer. As a general rule, these localities are a long way from a repair shop or supply house and the financial loss entailed by a burnt-out machine under these conditions is many times the value of the defective apparatus.

I do not suppose that even 1% of the dredging ground in existence is at present being mined; the process is still in its experimental stage. Although good results have been obtained so far, there is no doubt but that more advanced methods, both mechanical and electrical, will be discovered in the future to increase the efficiency of a process which is adding to the wealth of the world gold that would otherwise have remained in the ground where nature had placed it, viz.—gold dredging.

*The most efficient or critical peripheral speed of centrifugal pump in feet per minute

$$= \sqrt{\text{head in feet}} \times 8 \times 60$$

SINKING A SHAFT IN QUICKSAND BY THE FREEZING PROCESS.

The sinking of a shaft in quicksand is described in "The Iron and Coal Trade Review," June 5, 1903. The shaft is at the Laura and Vereeninging Colliery, in the Limburg province, Holland. It is 270 ft. deep to the coal measures, and 15 ft. inside diameter. We quote from our contemporary as follows:

At first a temporary shaft, with a diameter of about 30 ft., was sunk to a depth of 28 ft., and was provided with a provisional timbering consisting of planks and rings of channel iron. From the bottom of this shaft 24 boreholes were made, intended for the reception of the freezing tubes. These holes were sunk on the periphery of a circle with a diameter of 24 ft., so that the freezing tubes were at a distance of about 36 ins. from each other. They are of a diameter of 4 or 4½ ins., and accommodate the small descending pipes which only measure 1¼ ins. across. The latter communicate by means of valves with a distributing ring, and the former with a return ring introduced in order to be able to regulate the flow of the freezing solution. In the middle of each freezing tube there is inserted an elastic union which admits of its expansion and contraction with the fluctuations of the temperature. The freezing liquor, which consists of a solution of chloride of magnesium, leaves the refrigerators at a temperature of 20° C., and passes into the distributing ring, then down the small descending pipe as far as the bottom of the freezing tube, in order to ascend up the annular space between the pipe and the tube, till it reaches the return ring, from which it is conveyed by a pipe back again to the refrigerators, where the temperature is once more lowered. By means of the various valves any one of the freezing tubes can be cut out of the system. This operation is rendered necessary when one of them becomes stopped up or leaks.

For the production of the cold there was a double compressor of which the piston diameters were 5 ins. and 7½ ins., and the stroke 18 ins. This machine was driven by a steam engine through the medium of a belt. The carbonic acid was drawn in at a pressure of 220 lbs. per sq. in., and compressed to from 1,000 to 1,200 lbs. per sq. in. The compressed and consequently heated gas was next sent through a cooler, where it lost some of its heat, and then was passed into the two condensers in which it was liquefied, and finally the carbonic acid was conveyed through a third condenser to the refrigerators. Before entering the worms of this apparatus, it passed through an expansion valve, which suddenly brought the pressure from 1,100 lbs. down to about 200 lbs., causing a most intense absorption of heat, or, in other words, the production of intense cold. The refrigerator is a cylindrical vessel with a diameter of 66 ins., and a height of 110 ins., which is filled with the freezing mixture, consisting of a solution of magnesium chloride 30% strong. The spiral worm tube with the carbonic acid winds inside this vessel and causes the temperature of the solution to sink to 20° C., at which it is pumped into the distributing ring and from thence flows down the freezing tube.

The bore holes were carefully tested to see that they were perfectly vertical, for any serious deviation from the vertical would leave an unfrozen place in the wall of ice. Rotary borers were used and in three months' time the 24 holes were down, due to irregular working of the plant. It took eight weeks to freeze a wall of ice 20 ins. thick, and at the end of 12 weeks the ice wall was deemed thick enough to begin excavation. The frozen material was excavated with chisels and hammers, a soft unfrozen core considerably facilitating the work. This sinking (242 ft.) consumed 75 working days, or at the rate of 68 ins. per working day. In widening out the shaft in the coal-bearing strata a collapse occurred on the south side, filling the shaft with 40 ft. of sand and water. Water was then pumped into the shaft, filling it completely. Twelve more bore holes were driven in the shaft close to the sides, and six more holes on the south side of the shaft and 18 ft. from its center. This area was frozen and the work of excavation again begun, and 8½ months after the accident had occurred the bottom of the shaft was again reached.

MAGNETIC WATERS IN INDIANA.

By M. O. Leighton.*

To the majority of chemists and engineers the announcement of a magnetic water has a familiar sound of quackery usually associated with proprietary medicine circulars and widely advertised medicinal spring water. For some reason, well informed water chemists are nearly always skeptical when they are told that certain waters may impart magnetic properties to steel articles immersed therein. Many authorities have openly scoffed at such an idea.

Indeed, it is not long since that a leading article appeared in the Engineering News in which it was held that the power of imparting a so-called magnetic property to steel objects, which was claimed to exist in some waters, was probably fictitious, and, if it existed at all, it was probably due to the effect of the iron pipe or some similar conductor used in connection with the delivery of the spring water to convenient catch-basins.

There are three waters in the State of Indiana which actually impart magnetic powers to needles, knife blades, etc. One is at Cartersburg Springs, Hendricks Co., Ind., another is derived from a driven well at Lebanon, Ind., while the third is from a driven well at Fort Wayne.

These waters contain large proportions of carbon dioxide gas, which is slowly released upon exposure to the air. While the water retains this gas, it continues to impart magnetic properties to steel, but, as soon as the gas escapes, a heavy precipitate forms on the bottom of the container, consisting of magnetic oxide of iron. All magnetic properties are lost after this action takes place. The iron in solution is believed to be in the form of ferrous carbonate.

Experiments were made by the author at Cartersburg Springs. Knives and needles were carefully tested, both previous to reaching the springs and subsequent thereto, but before immersing in water, and no magnetic properties were present. After the knife had been immersed in the water for a period of five minutes, needles were readily

*Hydrographer U. S. Geological Survey, 263 Dearborn St., Chicago, Ill.

held, edge to point. Two needles immersed for two minutes were mutually attached by the points and held for a period of four minutes in the face of a strong breeze. The knife in question maintained its magnetic powers 30 hours after immersion.

There is no question concerning the magnetic property imparted to steel. It is not explained by the presence of the iron pipe, for in that immediate neighborhood there are many other springs and iron-cased wells which do not impart any such property to knife-blades and needles. The needle of the compass is readily deflected toward a body of this water.

The magnetic water at Lebanon comes from a driven well owned by the Big Four Railroad. When this well was first sunk and water was used in the locomotives, it was found to be the most corrosive water along the entire line; in fact, its use was almost fatal to the subsequent usefulness of the boiler. Examination by Dr. John N. Hurty, Chemist for the Big Four Railroad, showed absolutely no amount of any ingredients which would in any way account for this corrosive property. Upon examination in Lebanon, however, Dr. Hurty found that this water had the power of imparting magnetic properties to needles, axes and knives, and suggested that it be allowed to stand for a short period in a large tank before turning into the locomotives. This plan was adopted, and since that time the water has given no trouble on account of corrosive properties, although more recently the magnetic oxide is precipitated by introducing a jet of steam into the tank, rather than by resorting to the slower process.

From accurate descriptions, by reputable persons, received by the author, it is probable that the Fort Wayne water is more strongly magnetic than the two above mentioned, but as no opportunity has yet been afforded for any conclusive tests, the relative strength imparted is not determined.

Fig. 1. Fractures of Steel Bar Magnified Two Diameters.

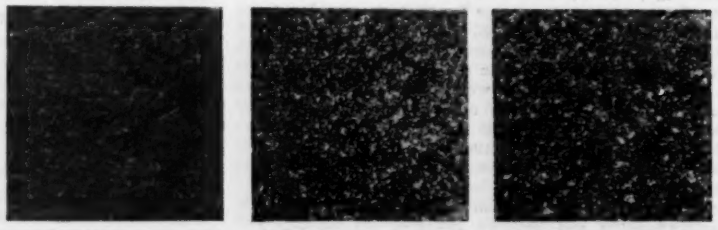


Fig. 2. Fractures of Steel Bar Magnified 50 Diameters.

Fig. 3. Original Fractured Surface of Bar.

1. 2. 3.

RESULTS OBSERVED UPON THE PHILADELPHIA & READING RY. IN CONNECTION WITH INSPECTION OF STEEL RAILS.*

By Robert Job.†

1. As to detection of coarse grain in steel rails by means of drop test:—(a) Our practice has proved beyond any question that the ordinary drop test of a weight of 2,000 lbs. falling 20 ft. upon butts about 4½ ft. long and 3 ft. between supports does not cause fracture of coarse-grained rails of 80 lbs. weight and over, excepting when brittleness due to other causes, such as piping, burning of steel, etc., is present.

(b) Large microstructure is due to slow undisturbed cooling from a high temperature and causes lessened ductility, and lessened strength under impact. Consequently it should be perfectly feasible so to proportion weight and height of drop as to produce fracture if grain exceed a certain size. Since service results show that resistance to wear, other things being equal, increases with decrease of size of grain, it follows that a test so arranged must be of great value as an indication of the service life of steel.

2. As to relation of microstructure to appearance of fractured surfaces.—(a) When the fractured surface appears coarse grained the microstructure is coarse grained.

(b) When the fractured surface appears fine grained the microstructure may or may not be fine grained. In other words, steel of coarse microstructure may be made to show a fairly fine-grained fracture under certain conditions of fracturing.

(c) When the microstructure is fine grained the fractured surface appears fine grained.

(d) To illustrate the relations existing at given points between appearance of grain in microstructure and upon fractured surfaces when the conditions attending the fractures are identical throughout, we took a steel slab about 6 ins. wide, 1½ ins. thick, and 2 ft. long, let one end remain in fire just below melting temperature for two hours, the other end being cool. The slab was then removed from the fire and allowed to cool in air, and a slot was cut about ¼-in. deep along the slab longitudinally. The slab was fractured longitudinally under the steam hammer and the surface was photographed, ½ size, Fig. 3, showing gradual change in size of grain from fine to coarse. (Metcalf's Experiment.)

The fracture of the steel was then photographed, magnified (2 diameters) Fig. 1, at points 1, 2 and 3 indicated upon Fig. 3. Microsections were taken from the same points, and appear (enlarged 50 diameters) in Fig. 2.

It will be seen that under these conditions of fracture there is a close general relation between the size of grain of the fractured surface, as seen by the eye and that shown in the microstructure at corresponding points. It should, however, be noted that when a number of rails from the same rolling are fractured, the surfaces may vary considerably from fine to coarse, although the general microstructure is usually quite constant because the latter is not subject to the varying conditions which attend the taking of different fractures.

The composition of the steel slab was as follows:

Carbon	.090
Manganese	.277
Phosphorus	.135
Sulphur	.106

2. As to relations between appearance of fracture, and poor service wear. In most cases rails of fairly solid steel fracturing in service or giving poor wear, have shown coarse grained fracture, indicating high finishing temperature. In relatively few cases the fracture has appeared fairly fine grained, but upon polishing and etching a section, the structure has always appeared coarse.

4. As to relations between coarse microstructure, and poor wearing qualities of rails in service. Upon a lot of 75,000 tons of 90-lb. rails observed during a period of five years, we have found fifteen times as many fractures in the case of rails having coarse microstructure, as in those of a much finer form. We have also invariably found with equal solidity of steel and with other conditions approximately equal a marked increase in rapidity of wear of steel of the coarse microstructure as compared with that of the finer.

5. As to relation between degree of shrinkage of rails after hot saws, and service life. (a) Rails with shrinkage of 7 ins. and over, on 30-ft. length have showed relatively large number of fractures and rapid wear.

(b) Rails with shrinkage of 6¼ ins. and less on 30-ft. length with rapid rolling and without holding between passes, have showed very largely decreased number of fractures together with increased capacity for wear.

6. As to relations between piping of rails and fractures in service and under drop test. A large majority of the rails fractured in service show piping to a considerable extent either directly upon the fractured surface or even when the cross section is polished and etched.

7. As to tests necessary to ensure tough, well wearing rails. Our practice during the past year has shown good results under the following conditions:

(a) Absence of brittleness is ensured in 90-lb. rails by drop test of 2,000 lbs. falling 23 ft. upon rail butts about 4½ ft. long, 3 ft. between supports, one test being made upon each heat, each test butt being taken from the top of an ingot.

(b) Toughness and moderately slow wear, if the steel is solid, is ensured by enforcing the following practice, viz., temperature of ingot or bloom to be such that with rapid rolling and without holding before or in the finishing passes or subsequently, and without artificial cooling after leaving last pass, the distance between hot saws shall not exceed 30 ft. 6 in. for a 30-ft. rail, or a proportionate distance for other lengths.

(c) Solidity of steel can be ensured only by careful inspection throughout the process of manufacture, especially as regards cropping.

(d) Suitable chemical composition is essential to the best service results. For 90-lb. rails we prefer about the following, with the ordinary rolling practice:

Carbon	0.53 to 0.63%
Phosphorus, below	.07
Sulphur, below	.07
Manganese	.90 to 1.20

*A paper presented at the meeting of the American Society for Testing Materials in Discussion of the Report on Specifications for Steel Rails.

†Chemist, Philadelphia & Reading, R. R.

AN EXCELLENT TYPE OF LAND PILE DRIVER.

Pile driving with a land driver is usually more expensive than with a scow driver because of the dimculty of shifting a land driver, as well as the time lost in blocking up or cribbing. Very frequently one sees a small land driver resting directly upon skids, along which it is pushed or slid "by main force and awkwardness." For the benefit of those who may not know how to make a simple land driver that can be quickly and easily shifted, we illustrate in Figs. 1 and 2 the general design of a land driver having exceedingly long leads (60 ft.). The feature of this driver that merits particular notice is the device for rolling it from place to place with ease.

Four 10-in. hard wood rollers, r, about 4 ft. long, are fastened to the bed plates, M, upon which the driver and its engine rest. These rollers, r, are provided with capstan holes, h, into which an iron bar may be inserted when it is desired to rotate them, capstan like. Each end of the roller is supported by an iron bearing, U, made of 1 × 3-in. bar iron, I, bent at each end into U-shape, as shown in Fig. 2. Wooden blocks, B, are bolted to the bed-plates, M, so as to reinforce these iron bearings. These rollers, r, in turn rest upon long transverse hard wood rollers, R, also provided with small capstan holes, h; and the rollers, R, ride upon stringers supported by blocking. In

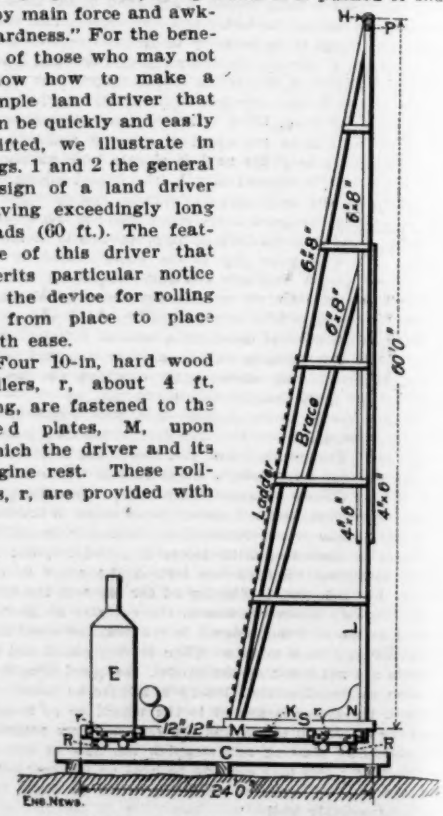


Fig. 1. Side View of Land Pile Driver on Rollers.

driving piles 2½ ft. c. to c., the crew of six men shift this large driver in just one minute, which those familiar with such work will recognize as being remarkably good time.

The hoisting engine is 10-HP., double drum, friction clutch, and is provided with one rope for hoisting piles, in addition to the hammer rope, hence the head-block has two sheaves, H. Two side guy-lines are fastened to trees, piles or other

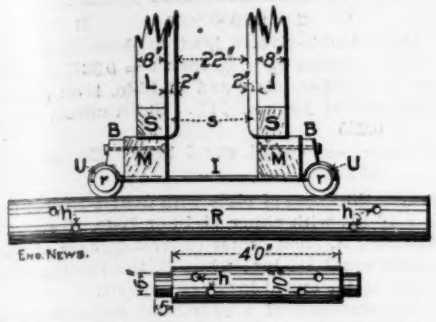


Fig. 2. Front View of Pile Driver Leads and Rollers.

anchorage, and pass through the pulleys, P, down to a cleat, K, around which the rope is wrapped. Occasionally one sees a land driver with the guy-lines fastened directly to the leads near the head block, in which case a man must be sent out to the anchor every time the driver is shifted, thus causing lost time for the whole crew. One often sees, also, a single drum engine used, and piles raised by the winch head. This also consumes more time than where a second drum is used. All in all, the driver shown in the accompanying cuts is one of the most commendable designs that we have seen for land pile driving.

THE RELATIVE STRENGTH OF WET AND DRY CONCRETE.*

By James W. Sussex.

The only reports of experiments on wet and dry concrete that the writer has been able to find are two given in the "Journal of the Western Society of Engineers," for October, 1900, and June, 1902. The former is a description of an experiment with wet and dry concrete by Mr. Irving Hitz, of the Chicago, Milwaukee & St. Paul R. R., and the latter is a report of an experiment conducted by Mr. H. W. Parkhurst of the Illinois Central R. R. Mr. Hitz's experiments consisted of two cubes, each containing about a yard of concrete, which were split open by wedges when one month old. The conclusions of Mr. Hitz were:

(1) A moderate excess of water is not injurious.
(2) More labor is necessary in mixing and placing a dry or moderately dry mixture than a wet one.
(3) It is impossible to get a compact concrete without using what is theoretically an excess of water.

Mr Parkhurst's experiments consisted of three cubes of wet, dry and medium mixtures, each containing about a yard of concrete, split open by wedges at the age of one month. Mr. Parkhurst's conclusion was that a medium concrete is the strongest.

Owing to the lack of data on the subject and to the importance of knowing the effect of a deficiency, or an excess of water in concrete, the following experiments were made: Forty-five 6-in. cubes were made with three percentages of water and broken in a testing machine at the ages of seven days, one month, and three months. The amount of water used was: For dry concrete 6%, for medium 7.8%, and for wet 9.4%. The ingredients used were as follows: Chicago AA Portland cement, sand containing a small percentage of fine gravel, and crushed limestone which would pass through a sieve with 1-in. mesh. The constituents were mixed in the proportion of 1 volume of loose cement, 3 volumes of sand, and 6 of broken stone.

The plasticity of the concrete with the per cents of water was as follows: The dry concrete was about as moist as damp earth, and when squeezed in the hand the mortar would retain its shape, and when thoroughly tamped no free water would flush to the surface. The medium concrete was of such consistency that it would neither quake in handling nor when slightly tamped, and when well tamped water would flush to the surface and the concrete would quake. The wet concrete quaked in handling and could be tamped only slightly. The wet and dry mixtures fairly represent the range in practice, while the medium concrete fulfils the common specification that "the concrete shall be of such consistency that water will flush to the surface when well tamped, but will not quake in handling."

Table I. shows the crushing strength of the cubes at different ages. The light tamping consisted of ten blows of a 11½-lb. iron tamper dropped 6 ins., on each 1-in. layer. The hard tamping consisted of 20 blows per 1-in. layer with the same tamper dropped the same distance. The ratio of strength is computed for each set of wet, dry and medium mixtures having the same age and the same tamping, taking the medium concrete as unity in each case.

These experiments show that at an age of 3 months or more, wet concrete is stronger than either dry or medium concrete. For a short period of time a medium concrete gives the greatest strength, which fact shows that wet concrete acquires its strength more slowly than the dryer mixtures.

In breaking the cubes it was noticed that the wet concrete was distorted more than the dry before failure occurred, which fact shows that wet concrete is more elastic than dry concrete.

By using a rather wet mixture a compact mass can be obtained with comparatively little tamping, while a dry mixture on the other hand, does not make a compact mass with any amount of tamping.

In the table it is seen by comparing Nos. 37, 38 and 39 with Nos 40. 41, and 42, that wet concrete with light tamping is stronger than either dry or medium concrete with hard tamping. Therefore wet concrete is cheaper than dry concrete and is also better.

There is no condition of age or amount of tamping which shows dry concrete to be the strongest.

The conclusions of these experiments may be stated briefly as follows:

(1) Dry concrete should never be used under any circumstances;
(2) Medium concrete may be used where immediate strength is desired;
(3) Wet concrete is stronger than either dry or medium, at any age over three months.

TABLE I.—Crushing Strength of 6-in. Concrete Cubes, Mixed with Different Percentages of Water.

Reference No.	Age.	Plasticity.	Tamping.	Crushing strength in lbs. Each.	Crushing strength in lbs. Mean.	Ratio of strength
1	7 days.	Dry.....	Light.	46,800		
2	"	"	"	41,700	43,400	0.533
3	"	"	"	41,800		
4	"	Medium.	"	86,600		
5	"	"	"	80,600	82,400	1.000
6	"	"	"	80,000		
7	"	Wet.....	"	38,300		
8	"	"	"	36,600	37,000	0.462
9	"	"	"	37,900		
10	"	Dry.....	Hard.	46,300		
11	"	"	"	46,500	48,300	1.007
12	"	"	"	52,000		
13	"	Medium.	"	46,800		
14	"	"	"	51,100	47,800	1.000
15	"	"	"	45,500		
16	1 mon..	Dry.....	Light.	55,000		
17	"	"	"	63,000	63,000	0.764
18	"	"	"	71,100		
19	"	Medium.	"	86,600		
20	"	"	"	80,000	82,400	1.000
21	"	"	"	80,600		
22	"	Wet.....	"	85,200		
23	"	"	"	79,500	80,300	0.973
24	"	"	"	76,300		
25	"	Dry.....	Hard.	67,400		
26	"	"	"	70,800	70,600	0.765
27	"	"	"	73,800		
28	"	Medium.	"	95,200		
29	"	"	"	91,800	92,300	1.000
30	"	"	"	90,000		
31	3 mos..	Dry.....	Light.	87,100		
32	"	"	"	88,400	89,800	1.160
33	"	"	"	94,000		
34	"	Medium.	"	76,000		
35	"	"	"	76,300	77,400	1.000
36	"	"	"	80,900		
37	"	Wet.....	"	110,700		
38	"	"	"	110,400	109,300	1.412
39	"	"	"	106,700		
40	"	Dry.....	Hard.	90,000		
41	"	"	"	92,400	93,600	1.005
42	"	"	"	98,400		
43	"	Medium.	"	99,300		
44	"	"	"	90,500	93,100	1.000
45	"	"	"	89,500		

*Abstract of thesis for degree of B.S. in civil engineering from the University of Illinois; published in the "Technograph" for 1903.

RAPID RIVER SOUNDING ON THE SURVEY OF THE DANUBE.

A rather special method of carrying on extensive series of river soundings has been used for some time by the Austrian government engineers in the hydrographic survey of the Danube. This work is for the purpose of obtaining topographic maps on a scale of 1 : 2,000 (something less than 200 ft. to the inch), and it is of course desirable to get the underwater contours as accurate as possible. This means soundings very closely spaced and accurately oriented. In arranging a system of working it was essential to provide for maximum rapidity (and consequent cheapness) of continuous series of soundings forming a very close network. The method used is as follows:

The sounding boat is rowed across the river on successive parallel courses, spaced as near to each other as the local conditions may require. A leadsman and a recorder in the boat make and record soundings as rapidly as they can work. Meanwhile the course of the boat is determined by stadia readings from an instrument on shore, a stadia rod being fixed to the boat. In practice it is found that the sounding party can make one observation every 5 to 10 seconds, while the instrument man and his recorder require 15 to 20 seconds for each reading of stadia and horizontal angle. The intermediate soundings must therefore be interpolated along the boat's course as mapped from the instrument readings. For this purpose the time of each sounding and of each stadia observation is noted, by the respective recorder, and on the assumption that the speed of the boat is uniform the time-intervals between observations are proportional to their space-intervals, so that the soundings may be located with sufficient accuracy. While otherwise perfectly satisfactory, the method is at fault in requiring a time-reading for each observation, which makes the work considerably slower than it would be if the clock reading could be dispensed with. Furthermore, the computation and plotting of the time-intervals makes the office work quite tedious.

Stadia Rod Used on Danube River Survey.

Recently a recording chronograph has been applied to this work, with good results. The recorder merely presses a button at the time the observation is made, whereby a pencil pressed against a moving strip of paper is moved momentarily so as to make a notch in the line; the time-intervals are then represented directly by the distances between successive marks. The chronographs used by the sounding party and the instrument party run at the same rate of speed, and are started and stopped simultaneously, on signal. The time-strips from the two machines can therefore be superposed, thereby directly locating the soundings with respect to the instrument observations. The soundings are then readily located on the map by spacing them in proper proportion between the points of the boat's course which were located by instrument. In case the speed of the boat and the rate of speed of the moving strip bear such a relation to each other that plotted distances along the course correspond exactly with the time-intervals on the chronograph strip, the office work is still further simplified, as the intermediate soundings may then be directly transferred from the time-strip to the map.

In the chronographs which were constructed for this work the circumference of the drum was made large enough to accommodate the longest single course which would likely be encountered. For successive courses of soundings the pencil is moved along the drum a short distance (4 mm.), the drum being brought back to zero for each course and started synchronously with the drum on the other instrument. The length of the drum is sufficient to take 30 courses of soundings, which represents the maximum day's work of a sounding party. Each day's time observations are therefore recorded on a single sheet.

In order to locate each course from a single instrument station, long stadia sights, frequently up to 400 m. long, became necessary. A special rod was designed for the work, as the ordinary rod was extremely difficult to read at these distances, especially since the reading must be done very quickly. The rod used is shown in the accompanying cut. It has decimeter divisions marked by black cross-strokes, and no smaller divisions except that the half-decimeters are marked by a row of dots whose number represents the whole meters from the foot of the rod. Half and full meters are marked by red lines, and the figures denoting the full meters are also red. The figures used are quite plain and as large as possible (each figure about 3½ ins. high) as the cut clearly shows. The rod is used with an in-

verting telescope, for maximum brightness of field, and must therefore be held inverted. The instrument man follows the boat with the telescope, keeping the lower wire on the end of the rod, and reading the other wire at the moment the vertical wire coincided with the rod. The river shore was usually flat enough to make the vertical angles of the stadia sights inappreciably small.

The general method of work outlined above was originally devised by Mr. R. Halter; the recording chronograph was applied to the work by Mr. R. Reich, who describes its use in a recent number of the "Zeitschrift des Oesterreichischen Ingenieur und Architeckten Vereins." The chronographs were made by Harpner & Ganser, of Vienna.

A TABLE FOR PITCH AND EFFICIENCY OF RIVETED JOINTS.

By Philip B. Hill.*

The accompanying table has been prepared to facilitate the design of a riveted joint for any given case. It gives, for any thickness of plate and every suitable size of rivet, the pitch of the rivets, the spacing of the rows (in joints with more than one row of rivets) and the efficiency of the joint. The rivet pitches given in the table are such that the ultimate tensile strength of the plate between the rivet holes, at 60,000 lbs. per sq. in., is equal to the ultimate shearing strength of the rivets, at 38,000 lbs. per sq. in. The efficiency of the joint is the quotient obtained by dividing the distance between edges of rivet holes by the rivet pitch c. to c. of holes, or the ratio between the net and gross section of plate.

CALCULATION OF TABLE.—The areas corresponding to the diameters of rivet holes were taken from a table of areas in Trautwine and multiplied by 38,000. The areas and products in thousands appear in lines 4 and 5. The thicknesses of plate were multiplied by 60,000, and the products checked by subtraction. For each thickness of plate, the former products were divided by the latter (for all practical sizes of rivets); the quotient is the distance from edge to edge of rivet holes of a single-riveted joint. Twice and three times these values gave the distances between edges of rivet holes for double and triple-riveted joints, respectively. To these values were added the corresponding diameters of rivet holes, to obtain the pitch, or distance, from center to center of rivets.

The lengths of net section of plate between the edges of rivet holes in inches and decimals, as obtained, were given values to the nearest sixteenth of an inch and added to the diameters in sixteenths of an inch and checked by adding the decimal equivalents of the diameters to the original values.

The efficiency of the joint was obtained by dividing the length of plate between the edges of rivet holes by the pitch of rivets, both expressed decimally. For example, take a 5-16-in. plate and 11-16-in. rivet, diameter of hole ¾-in. (area of hole 0.442). Then $0.442 \times 38{,}000 = 16{,}796$, given in the table as 16,800; $5\text{-}16 \times 60{,}000 = 18{,}750$. Dividing the first by the second, $16{,}800 \div 18{,}750 = 0.8961$-in., the distance edge to edge of rivet holes. Instead of this use the nearest sixteenth, which is ⅞-in., or 0.875. Adding the diameter of the rivet hole, ¾-in., or 0.75, gives for the pitch 1⅝-in., or 1.625. The course of the calculation for single, double and triple-riveted joints is summarized by the following schedule:

⅝-in. plate; ¹¹⁄₁₆-in. rivet; ¾-in. rivet hole.

Clear spacing:	—Single.—	—Double.—	—Triple.—
Exact	0.8961	1.7922	2.6883
Use	⅞ = 0.875	$1^{13}/_{16}$ = 1.813	$2^{11}/_{16}$ = 2.688
Add rivet diam.	¾ = 0.750	0.750	0.750
Pitch, c. to c.	1⅝ = 1.625	$2^{9}/_{16}$ = 2.563	$3^{7}/_{16}$ = 3.438
Efficiency	0.5382	0.7074	0.7817

All calculations, except the multiplication of areas by 38,000 and thicknesses by 60,000, were performed on a Schuerman computer, reading to four places. The calculations of the values of efficiency were checked by noting that the values increased by practically a constant quantity for a uniform increase of diameter of rivets and a constant thickness of plate; also that one-half the difference between the efficiencies of a single and double-riveted joint, added to the efficiency of the latter, gave the efficiency of a triple-riveted joint. The latter check came out usually within 1%, the discrepancy being due to the use of even sixteenths-inch for the pitch instead of the exact decimal value. As it was not attempted to give values closer than 1% the method served as a satisfactory check.

The areas of rivet holes and the products of these areas by 38,000 are given in the table to enable the pitch for any tensile strength other than 60,000 to be rapidly calculated.

USE OF TABLE.—To illustrate the use of the table, suppose we require the thickness of a boiler shell to withstand a certain pressure, or have a stand-pipe of a certain diameter and height to design. In the former case, the required thickness of plate is represented by the formula,

$$t = \frac{P D F}{2 T E},$$

where

t = thickness of plate, in inches,
P = steam pressure, in lbs. per sq. in.,
D = inside diameter of shell, in inches,
F = factor of safety,
T = ultimate tensile strength of plate, in lbs. per sq. in.,
E = efficiency of joint.

For example, take a boiler 60 ins. in diameter, to withstand a steam pressure of 100 lbs. per sq. in., with a factor of safety of 4½, the ultimate strength of the steel shell being 60,000 lbs. per sq. in. In this case the above formula gives,

$$t = \frac{100 \times 60 \times 4.5}{2 \times 60{,}000 \times E} = \frac{0.225}{E}.$$

For a double-riveted joint, to obtain an approximate t, assume E = ⅔, then $t = 0.3375$ = 11-32-in. A 11-32-in. plate and 13-16-in. rivet gives an efficiency of joint of 71%. Substituting, we get

$$t = \frac{0.225}{0.71} = 0.313,$$ say 5 16-in. Therefore, the proper thickness of plate with double-riveted joint is 5-16-in., with either 11-16 or ¾-in. rivets. The distance, A, from center of rivet to edge of plate, would equal 1¼ ins., while B, the spacing of the rows, is 1¾ and 2 ins., respectively.

The standard of a prominent steam boiler inspection and insurance company for 5-16-in. plate and ¾-in. rivets, double lap-riveted joint, is: Pitch = 2⅞ ins., B = 1 15-16 ins., A = 1 7-32 ins., E = 72%. The table gives: Pitch = 2 15-16 ins., B = 2 ins., A = 1¼ ins., E = 72%.

For stand-pipes or tanks the required thickness of shell at any horizontal plane below the maximum water level is given by the formula:

$$t = \frac{62.5 D H F}{24 T E},$$ where

D = diameter of tank, in feet,
H = head of water above the section in question, in feet,
F = factor of safety,
T = ultimate tensile strength of steel, in lbs. per sq. in.,
E = efficiency of joint.

Having selected the diameter and height of tank, considering it full, an approximate value of t can be obtained by considering all the vertical seams double-riveted with E = ⅔; then taking a factor of safety of 4 and an ultimate tensile strength of plate of 60,000 lbs. per sq. in., the formula becomes:

$$t = 0.00026 D H.$$

If we take H in 5-ft. intervals below the top, we obtain the approximate thickness of shell at these points. By a few trials, the correct thickness of plate and the best type of joint and pitch are obtained. The ring seams, which are as a rule single-riveted, are generally given pitches greater than those given in the table for a single-riveted joint, as these joints require an efficiency of only about 40%.

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The pitches below are calculated so that the ultimate tensile strength of the net section of plate at 60,000 lbs. per sq. in. equals the ultimate shearing strength of rivets at 38,000 lbs. per sq. in., the diameter of rivet holes being used to obtain the rivet area. The efficiency of joint is the ratio of the net to the gross section of plate.

THICKNESS OF PLATE	5/16 — .1875				¼ — .25				9/32 — .2813				5/16 — .3125				11/32 — .3438				⅜ — .375				13/32 — .4063				7/16 — .4375			
DIAMETER OF RIVETS	⅜	7/16	½	9/16	7/16	½	9/16	⅝	5/16	11/16	¾	½	9/16	⅝	11/16	¾	½	9/16	⅝	11/16	⅝	11/16	¾	13/16	¾	13/16	⅞	15/16	⅞	15/16	1	1/16
DIAMETER OF RIVET HOLE	7/16	9/16	9/16	11/16	9/16	9/16	11/16	11/16	11/16	13/16	13/16	9/16	11/16	11/16	13/16	13/16	9/16	11/16	11/16	13/16	11/16	13/16	13/16	15/16	13/16	15/16	15/16	1 1/16	15/16	1 1/16	1 1/16	1 3/16
AREA OF RIVET HOLE	150	196	249	307	196	249	307	371	442	518		249	307	371	442	518	249	307	371	442	307	371	442	518	601	442	518	601	690	518	601	690	785
AREA × 38000	5700	7448	9462	11 67	7448	9462	11 67	1410	1680	1968		9462	11 67	1410	1680	1968	9462	11 67	1410	1680	1168	1410	1680	1968	2284	1680	1968	2284	2622	1968	2284	2622	2983
P. SINGLE RIVETED PITCH	15/16	1 1/16	1 1/16	1 1/16	1	1 1/16	1 1/16	1 1/8	1 1/8	1 1/8		1 1/8	1 1/16	1 1/16	1 1/8	1 1/8	1 1/8	1 1/16	1 1/16	1 1/8	1 1/16	1 1/16	1 1/8	1 1/8	2	1 1/16	1 1/8	1 1/8	2	1 1/16	1 1/8	1 1/8	2 1/8
EFFICIENCY OF JOINT	58	58	61	63	50	53	58	60	62			50	52	54	57	59	47	50	52	54	47	50	52	54	56	50	52	53	56	50	52	53	56
P. DOUBLE RIVETED PITCH	1 7/16	1 7/16	2 1/16	2 1/16	1 1/2	1 1/16	2 1/16	2 1/16	3	3 1/16		1 1/16	2	2 1/16	2 1/16	3 1/16	1 1/16	1 1/16	2 1/16	2 1/16	1 1/16	2 1/16	2 1/16	2 1/16	3 1/16	2 1/16	2 1/16	2 1/16	3 1/16	2 1/16	2 1/16	2 1/16	3 1/16
EFFICIENCY OF JOINT	70	72	75	77	67	69	71	73	75	76		67	69	71	73	74	64	67	69	71	64	67	68	70	71	67	68	69	71	67	68	69	65
P. TRIPLE RIVETED PITCH	1 15/16	2 1/16	2 1/16	3 1/16	2	2 1/16	2 1/16	3 1/16	4 1/16	4 1/16		2 1/16	2 1/16	3 1/16	3 1/16	4 1/16	2 1/16	2 1/16	2 1/16	3 1/16	2 1/16	2 1/16	3 1/16	3 1/16	4 1/16	3	3 1/16	3 1/16	4 1/16	3 1/16	3 1/16	3 1/16	4 1/16
EFFICIENCY OF JOINT	78	80	82	83	75	77	79	80	82	83		75	77	78	80	81	75	75	77	78	79	75	75	77	78	79	75	76	77	79	75	76	78

THICKNESS OF PLATE	15/32 — .4688			½ — .5				17/32 — .5313				9/16 — .5625				19/32 — .5938				⅝ — .625				11/16 — .688				¾ — .75			
DIAMETER OF RIVETS	15/16	7/8	15/16	1	15/16	7/8	15/16	1	1 1/16	7/8	15/16	1	1 1/16	7/8	15/16	1	1 1/16	7/8	15/16	1	1 1/16	7/8	1	1 1/16	1 1/8	1 1/16	1	1 1/16	1 1/8	1 1/16	1	1 1/16	1 1/8
DIAMETER OF RIVET HOLE	1 1/16	15/16	1 1/16	1 1/16	1 1/16	15/16	1 1/16	1 1/16	1 1/16	15/16	1 1/16	1 1/16	1 1/16	15/16	1 1/16	1 1/16	1 1/16	15/16	1 1/16	1 1/16	1 1/16	15/16	1 1/16	1 1/16	1 1/16	1 1/16	1 1/16	1 1/16	1 1/16	1 1/16	1 1/16	1 1/16	1 1/16
AREA OF RIVET HOLE	601	690	785	518	601	690	785	601	690	785	887	601	690	785	887	994	690	785	887	994	690	785	887	994	1 11	785	887	994	1 11	785	887	994	1 11
AREA × 38000	2284	2622	2983	1968	2284	2622	2983	2284	2622	2983	3371	2284	2622	2983	3371	3772	2622	2983	3371	3777	2622	2983	3371	3777	4218	2983	3371	3777	4218	2983	3371	3777	4218
P. SINGLE RIVETED PITCH	1 11/16	1 7/8	2 1/16	1 1/2	1 7/8	1 15/16	2	1 15/16	1 15/16	1 15/16	2 1/8	1 15/16	1 15/16	1 15/16	2 1/16	2 1/4	1 15/16	1 15/16	2	2 1/16	1 15/16	1 15/16	1 15/16	2 1/16	2 1/16	1 15/16	1 15/16	1 15/16	2 1/16	2 1/16	1 15/16	1 15/16	1 15/16	2 1/16
EFFICIENCY OF JOINT	48	50	52	40	46	48	50	46	46	48	50	44	44	47	48	50	46	46	47	49	42	44	45	47	49	43	43	45	46	41	41	42	44
P. DOUBLE RIVETED PITCH	2 1/16	2 1/16	2 1/16	2 1/16	2 1/16	2 1/16	3	2 1/16	2 1/16	2 1/16	3 1/16	2 1/16	2 1/16	2 1/16	3 1/16	3 1/16	2 1/16	2 1/16	2 1/16	3 1/16	2 1/16	2 1/16	2 1/16	3 1/16	3 1/16	2 1/16	2 1/16	2 1/16	3 1/16	3 1/16	2 1/16	2 1/16	2 1/16	3 1/16
EFFICIENCY OF JOINT	65	67	68	52	63	65	67	62	63	65	67	61	63	64	65	67	63	63	64	65	59	61	63	64	65	59	61	62	63	57	58	60	61
P. TRIPLE RIVETED PITCH	3 1/16	3 1/16	4 1/16	2 1/16	3 1/16	3 1/16	4	3	3 1/16	3 1/16	4 1/16	2 1/16	3 1/16	3 1/16	4 1/16	4 1/2	3 1/16	3 1/16	3 1/16	4 1/16	3 1/16	3 1/16	3 1/16	4 1/16	4 1/2	3 1/16	3 1/16	3 1/16	4 1/16	3	3 1/16	3 1/16	4 1/16	4
EFFICIENCY OF JOINT	74	75	76	71	72	74	75	71	72	74	75	70	71	72	74	75	72	72	73	74	69	70	72	73	74	69	70	71	72	67	68	69	70

THICKNESS OF PLATE	5/16	¼	5/16	3/8	7/16	½	9/16	5/8	11/16	¾
A. DISTANCE FROM CENTER OF RIVETS TO EDGE OF PLATE	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛

B. TRANSVERSE PITCH OF RIVETS FOR DOUBLE AND TRIPLE RIVETED JOINTS = ⅝ OF ABOVE PITCHES TO NEAREST 1/16 INCH

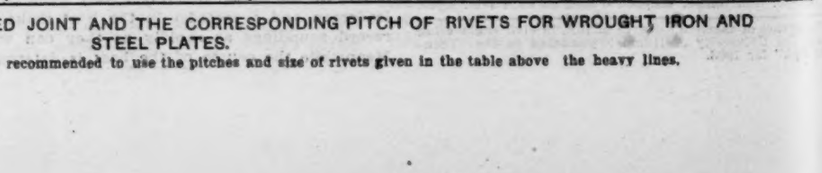

TABLE OF MAXIMUM EFFICIENCY OF RIVETED JOINT AND THE CORRESPONDING PITCH OF RIVETS FOR WROUGHT IRON AND STEEL PLATES.

For steam and water-tight joints it is recommended to use the pitches and size of rivets given in the table above the heavy lines.